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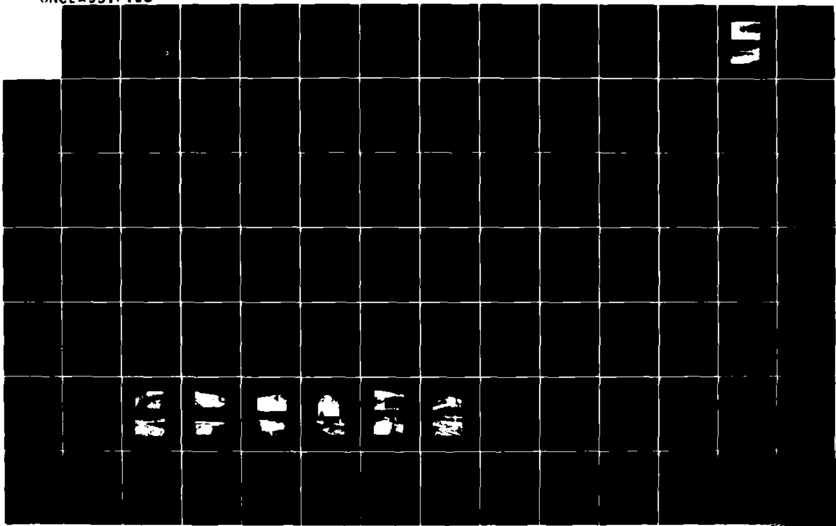

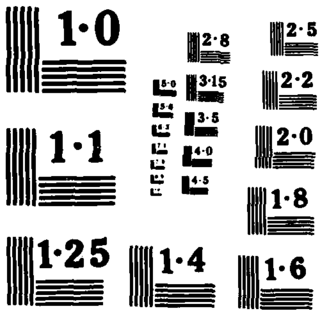
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PROVIDENCE RIVER BASIN
WOONSOCKET, RHODE ISLAND

HARRIS POND DAM
RI 03901

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

FEBRUARY, 1979

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER RI 03901	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) Harris Pond Dam NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS		5. TYPE OF REPORT & PERIOD COVERED INSPECTION REPORT 6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS DEPT. OF THE ARMY, CORPS OF ENGINEERS NEW ENGLAND DIVISION, NEDED 424 TRAPELO ROAD, WALTHAM, MA. 02254		12. REPORT DATE February 1979 13. NUMBER OF PAGES 84
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)		15. SECURITY CLASS. (of this report) UNCLASSIFIED 15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

APPROVAL FOR PUBLIC RELEASE: DISTRIBUTION UNLIMITED

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

Cover program reads: Phase I Inspection Report, National Dam Inspection Program; however, the official title of the program is: National Program for Inspection of Non-Federal Dams; use cover date for date of report.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS, INSPECTION, DAM SAFETY,
Providence River Basin
Woonsocket, Rhode Island
Mill RIver

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This
~~The~~ dam is a zoned earth embankment about 1200 ft. long with a maximum height of about 40 ft. The dam is intermediate in size with a high hazard potential. Because of this the test flood is the full PMF. Both the dam and its appurtenant structures are judged to be in generally good condition. There are various remedial measures which must be undertaken by the owner.

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

REPLY TO
ATTENTION OF:
NEDED

SEP 24 1979

Honorable J. Joseph Garrahy
Governor of the State of Rhode Island
and Providence Plantations
State House
Providence, Rhode Island 02903

Dear Governor Garrahy:

I am forwarding to you a copy of the Harris Pond Dam Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. This report is presented for your use and is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. A brief assessment is included at the beginning of the report. I have approved the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is a vitally important part of this program.

A copy of this report has been forwarded to the Department of Environmental Management, the cooperating agency for the State of Rhode Island. In addition, a copy of the report has also been furnished the owner, City of Woonsocket, Rhode Island.

Copies of this report will be made available to the public, upon request, by this office under the Freedom of Information Act. In the case of this report the release date will be thirty days from the date of this letter.

I wish to take this opportunity to thank you and the Department of Environmental Management for your cooperation in carrying out this program.

Sincerely,

MAX B. SCHEIDER
Colonel, Corps of Engineers
Division Engineer

Incl
As stated

HARRIS POND DAM

RI 03901

PROVIDENCE RIVER BASIN
WOONSOCKET, RHODE ISLAND

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

NATIONAL DAM INSPECTION PROGRAM
PHASE I INSPECTION REPORT

Identification No.:	RI 03901
Name of Dam:	Harris Pond Dam
Town:	Woonsocket
County and State:	Providence County Rhode Island
Stream:	Mill River
Date of Inspection:	27 September 1978

BRIEF ASSESSMENT

Harris Pond Dam is a zoned earth embankment about 1,200 ft. long, with a maximum height of about 40 ft. A low auxiliary dike about 300 ft. long is constructed across the right abutment. The present dam was reconstructed in 1969, incorporating the remains of a nineteenth century dam which was breached in 1955. The dam is operated as a water supply facility for the City of Woonsocket.

The spillway has a 150 ft. long ungated ogee crest which discharges into a stepped chute and stilling pool. Stored water is released to a pumping station via a 20 in. dia. pipe from a wet well shaft, and a 36 in. dia. outlet pipe discharges into a downstream channel.

Maximum storage capacity of the reservoir to top of dam is about 2,850 acre-ft. and the drainage area is 32.5 square miles. The reservoir is about 1.85 miles long with a surface of 100 acres at spillway crest elevation. Based on both height and capacity criteria, the dam is classified as intermediate in size. Because of the threat to life and property which would result from the dam being breached, particularly in the Social area of Woonsocket, it has been classified as having a high hazard potential. Based on intermediate size and high hazard, the test flood is the full PMF.

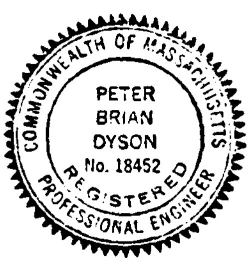
Some seepage was observed along the downstream toe of the dam, and there is some minor erosion of the upstream slope. Brush and light tree growth are established in the outlet channel, and there is also some light brush on the upstream slope. One of the outlet control valves is inoperable. Both the dam and its appurtenant structures are judged to be in generally good condition.

The test flood inflow is 17,500 cfs., while the test flood outflow is 17,200 cfs. The test flood surcharge elevation of 175.9 would overtop the lowest point of the sloping dike by 3.7 ft., with 2,900 cfs. of the outflow being discharged over the dike and 14,400 cfs. being released through the spillway. A 0.5 PMF event would also overtop the dike by a maximum of 1.3 ft. The spillway chute walls would be completely overtopped by the test flood outflow and partially overtopped by a 0.5 PMF outflow. The spillway would pass about 32 percent of test flood without overtopping the dike.

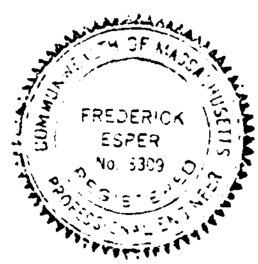
Within one year after receipt of this Phase I Inspection Report, the owner, the City of Woonsocket, should retain the services of a registered professional engineer to make further hydrologic and hydraulic evaluations, and should implement the results. These investigations should cover the potential overtopping of the dike and right abutment, whether the dike should be raised, and the spillway flow conditions below the crest and through the chute.

The owner should also implement the following measures: (1) repair minor erosion and restore displaced riprap on the upstream slope; (2) control growth on the upstream slope and in the outlet channel; (3) monitor seepage and toe drain discharges monthly during periods of high reservoir level; (4) repair the unserviceable 24 in. dia. outlet valve; (5) develop a formal surveillance and warning plan; and (6) institute procedures for a biennial periodic technical inspection.

Peter B. Dyson
Project Manager



Frederick Esper
Frederick Esper
Vice President



This Phase I Inspection Report on Harris Pond Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgement and practice, and is hereby submitted for approval.

Joseph A. McElroy

JOSEPH A. MCELROY, MEMBER
Foundation & Materials Branch
Engineering Division

Carney M. Terzian

CARNEY M. TERZIAN, MEMBER
Design Branch
Engineering Division

Joseph W. Finegan, Jr.

JOSEPH W. FINEGAN, JR., CHAIRMAN
Chief, Reservoir Control Center
Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:

Joe B. Fryar

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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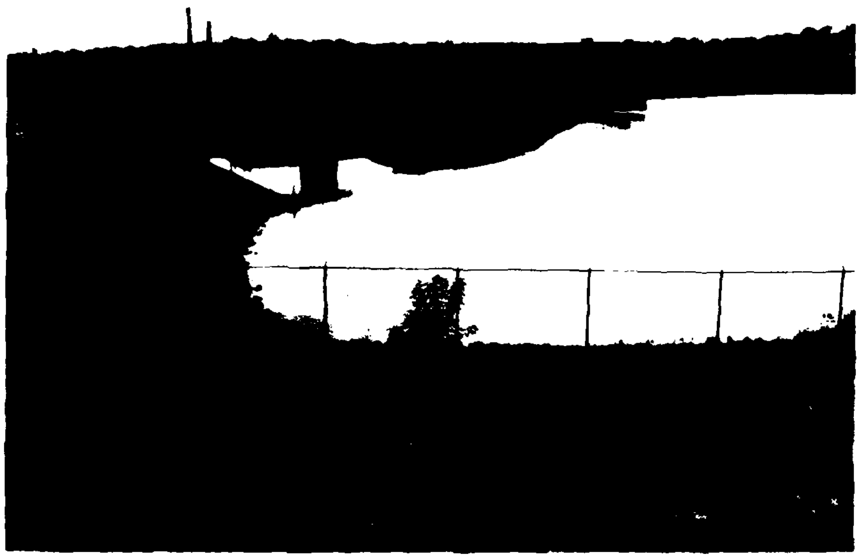
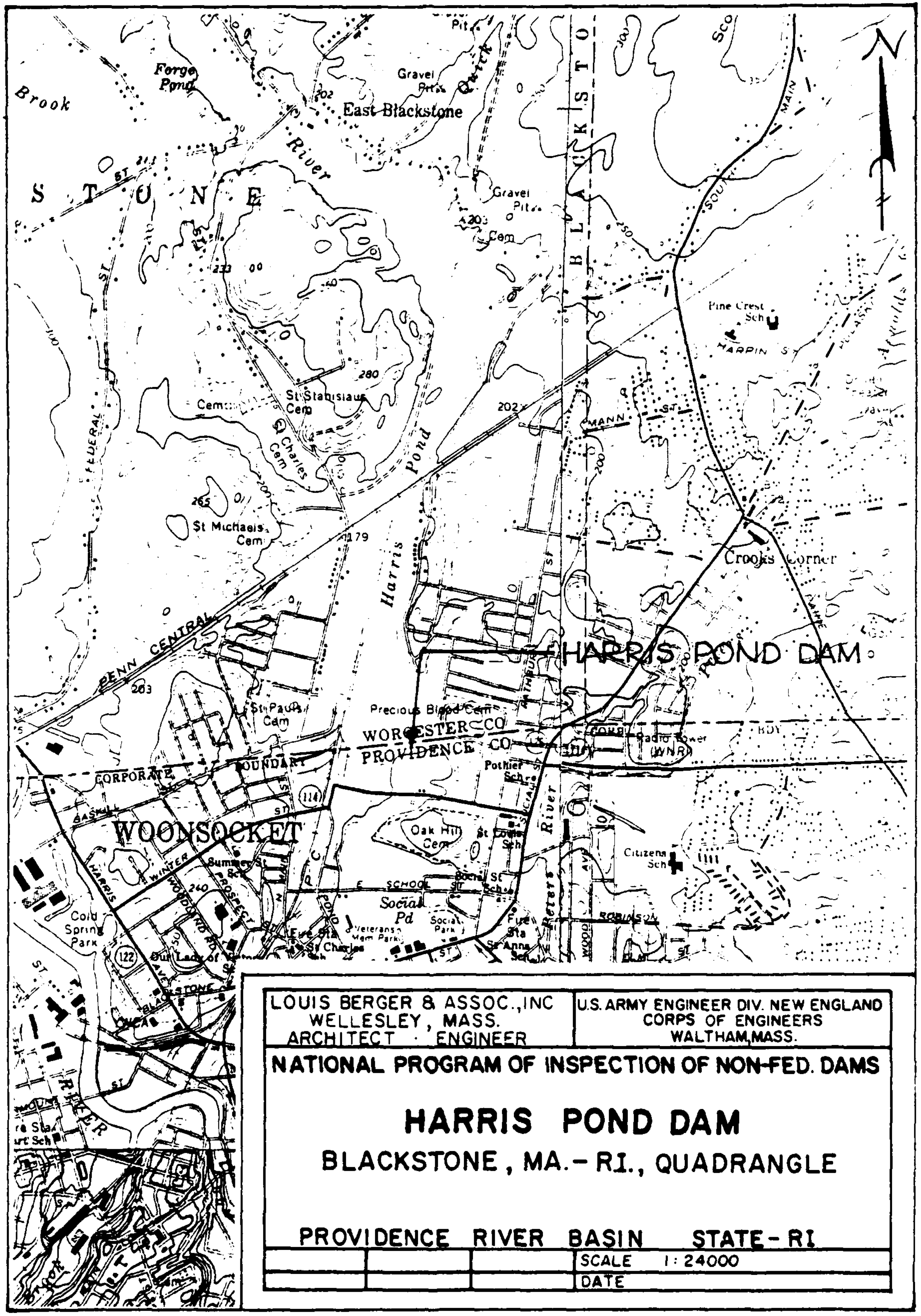
HARRIS POND DAM



Overview from Precious Blood Cemetery at left abutment

Overview from right abutment



LOUIS BERGER & ASSOC., INC WELLESLEY, MASS. ARCHITECT · ENGINEER	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

HARRIS POND DAM

BLACKSTONE, MA.-RI., QUADRANGLE

PROVIDENCE RIVER BASIN STATE-RI

SCALE 1:24000

DATE

PHASE I INSPECTION REPORT

HARRIS POND DAM RI 03901

SECTION 1 - PROJECT INFORMATION

1.1 General

a. Authority

Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a national program of dam inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Louis Berger & Associates, Inc. has been retained by the New England Division to inspect and report on selected dams in the State of Rhode Island. Authorization and notice to proceed was issued to Louis Berger & Associates, Inc. under a letter of 24 August 1978 from Ralph T. Garver, Colonel, Corps of Engineers. Contract No. DACW33-78-C-0371 has been assigned by the Corps of Engineers for this work.

b. Purpose

1. Perform technical inspection and evaluation of non-Federal dams to identify conditions which threaten the public safety and thus permit correction in a timely manner by non-Federal interests.

2. Encourage and assist the States to initiate quickly effective dam safety programs for non-Federal dams.

3. To update, verify and complete the National Inventory of Dams.

1.2 Description of Project

a. Location

Harris Pond Dam is located on Mill River at the Massachusetts-Rhode Island state boundary. The main dam is in the City of Woonsocket, Providence County, Rhode Island, although part of the downstream

The profile along the crest of the dam and dike is not horizontal, but ramped each way from the spillway to the dam abutments. This profile is delineated on Figure 1, Sheet D-1, Appendix D, and on the drawings in Appendix B. The crest of the dam varies from elevation 180.3 to 177.3, and the crest of the dike varies from elevation 177.3 to 172.2, the lowest point being at the right abutment. The minimum freeboard above normal storage for the main dam embankment is 9.8 ft.

The typical reconstructed cross section in the area encompassing the original dam has a 2½ horizontal to 1 vertical upstream slope and a 2 to 1 downstream slope. The upstream slope basically consists of a sloping impervious earthfill blanket placed against the pre-existing earth dam, which is covered with a 12 in. layer of bank run gravel and a 6 in. layer of screened gravel. These layers are in turn overlain by an 18 in. thick riprap over the entire upstream slope. The downstream slope consists of a pervious earthfill and the crest width is 35 ft. A stone gutter was incorporated along the toe of the downstream slope. A drain blanket and a toe drain collector system were installed at the toe of the downstream slope as shown on the contract drawings. The typical dam section in the east abutment breached area is a continuation of the typical embankment section previously described, except that a random pervious fill material was used to fill the breached gap.

A cut-off trench was provided beneath the impervious zone along the toe of the upstream slope of the embankment. This cut-off trench was extended either to bedrock or to a maximum depth of approximately 10 ft. for the entire length of the dam. The dike from the right end of the dam apparently had no cut-off. Where the cut-off trench did not contact rock, a considerable thickness of natural granular soil overlies the rock. Seepage from the reservoir would, therefore, be expected to go through this layer. Analytical studies were made in 1968 by the dam designer and seepage was predicted through this area. A toe drainage system was incorporated to intercept this seepage.

2. Spillway

The spillway is located on the knoll between the main embankment to the right and the refilled breach gap to the left. The spillway channel is directed along the downstream face of the knoll, its centerline making an angle of about 69 degrees with the axis of the dam. The spillway has a 150 ft. long ungated ogee crest converging to a 90 ft. wide, three-stepped rectangular concrete-lined chute, and then to a trapezoidal riprap-lined stilling pool. The ogee crest is at elevation 167.5; the floor of the riprapped basin is at elevation 132. The total length of the concrete chute is about 250 ft. The spillway was designed for a normal capacity of 8,500 cfs. at a surcharge head of about 6 ft., which is 1.3 ft. higher than the low point on the dike. The spillway capacity when the dike would start to be overtopped is 5,500 cfs.

The spillway ogee crest structure, chute walls and floor slabs were placed on natural ground, with a gravel blanket and open-jointed vitrified pipe underdrainage system.

3. Outlets

Outlet pipes at three selective levels are provided for releasing stored waters from the reservoir. Two 16 in. dia. cast iron inlet pipes, at elevations 160.5 and 153.0, lead into a wet well shaft where inflows are controlled by butterfly valves. A 20 in. dia. pipeline conveys these flows from the shaft to a pumping station situated to the right of the spillway. A 36 in. dia. low-level outlet pipe at elevation 144.5 leads from the reservoir to empty into a downstream channel, with control regulated by two 24 in. dia. butterfly valves located in the wet well shaft.

c. Size Classification

Harris Pond Dam varies between 38 ft. and 41 ft. in height above downstream river level, and impounds a normal storage of about 1,050 acre-ft. to spillway crest level and a maximum of about 2,850 acre-ft. to top of dam. In accordance with the size and capacity criteria given in Recommended Guidelines for Safety Inspection of Dams, the project falls into the intermediate category for both criteria and therefore is classified accordingly.

d. Hazard Classification

A breach failure of Harris Pond Dam would release water down the Mill River's rechannelized channel and through the flood control conduit constructed under the Social area of Woonsocket. The Mill River flood control conduit was designed for a discharge of 8,500 cfs, above which the inlet walls and dikes above the conduit entrance would be overtopped. Such overtopping would flood out the entire Social area behind the confining floodwall along the Blackstone River, causing extensive property damage and possible loss of life. In view of this, the dam has been classified as having a high hazard potential in accordance with the Recommended Guidelines for the Safety Inspection of Dams.

e. Ownership

Harris Pond Dam is owned by the City of Woonsocket, Rhode Island.

f. Operator

Makram H. Megalli, P. E.
Director of Public Works
City Hall
169 Main Street
Woonsocket, RI 02895

Hedley V. Patterson
Division Engineer

Russell S. Horne
Water & Sewer Superintendent

Telephone: (401) 762-6400

g. Purpose of Dam

Harris Pond is operated in conjunction with other facilities for water supply for the City of Woonsocket.

h. Design and Construction History

The present dam was constructed in 1968-69 on the site of a nineteenth century earth dam which was breached on 19 August 1955, during hurricane "Diane". The breach was about 100 ft. wide and included the spillway.

According to a report dated August 23, 1955 (Appendix B), the failure of the Spindleville Dam near Hopedale, Massachusetts, about 7 miles upstream, apparently began a chain reaction of dam failures down the Mill River.

No information has been recovered about the design and construction of the original dam in the nineteenth century. The files of the RI Department of Environmental Management show that in the 1930's the dam was owned by Woonsocket Rayon Co. and in the 1940's by Synthetic Yarns, Inc.

After the 1955 dam failure, the City of Woonsocket acquired the water rights from the previous owners. The present dam was designed by Metcalf & Eddy, Engineers (now Metcalf & Eddy, Inc., 50 Staniford St., Boston, MA 02114) for the city, and reconstruction incorporated the original earth dam into the new embankment. The normal reservoir level, however, is now 2 ft. lower than that maintained by the old spillway.

i. Normal Operational Procedure

There are no written operating procedures. According to Public Works Department personnel, Harris Pond is only utilized occasionally as a source of water. The pumps are tested once a year. If the water level falls below normal, the 6 in. dia. bypass valve is opened to provide some downstream flow.

1.3 Pertinent Data

a. Drainage Area

The drainage area above the Harris Pond Reservoir covers about 32.5 square miles, being about 14 miles long and an average of 2.3 miles wide. The area measures 3.8 miles at its widest point. Upstream from Harris Pond on the Mill River or its tributaries are the following impoundments: Forge Pond immediately upstream with a surface area of 16 acres at elevation 177; Lake Hiawatha about 1 mile upstream on Quick Stream with a surface area of 60 acres at elevation 229; Spindleville Pond about 6 miles upstream with a surface area of 12 acres at elevation 235; Hopedale Pond about 7½ miles upstream with a surface area of 92 acres at elevation 274; Fiske Millpond about 10½ miles upstream with a surface area of 16 acres at elevation 295;

and North Pond about 12½ miles upstream with surface area of 232 acres at elevation 348. A sketch of the area showing the location of the pondages and streams is shown on Sheet D-2, Appendix D.

The topography of the drainage area is mainly rolling hills, being generally wooded with occasional small swampy areas along the main stream course. The rim of the basin rises to an average of about 200 ft. above the stream valley, with individual hills rising to as much as 300 ft. above the valley floor. The watercourse upstream above Harris Pond Reservoir measures about 15.7 miles, with an average slope of about 13 ft. per mile. Except in the towns of Hopedale and Spindleville and in the areas surrounding North Pond, the area is sparsely developed, with residences scattered along the network of secondary roads in the area.

b. Discharge at Damsite

1. Outlet Works Conduit

The 36 in. dia. low-level outlet pipe can release about 125 cfs with reservoir at spillway crest level elevation 167.5. An average of about 80 cfs can be released through the range of reservoir levels from elevation 146 to 167.5. A discharge curve is shown on Sheet D-3, Appendix D.

2. Maximum Known Flood at Damsite

The maximum known flood at Harris Pond Dam occurred during the August 1955 storm, caused by a record rainfall amplified by the failure of the Spindleville Dam upstream. It was estimated that the flood inflow peaked at about 3,400 cfs and that the flow increased below the dam after the failure of the spillway and subsequent breaching.

3. Spillway Capacity at Top of Dam

The spillway was designed to pass about 8,500 cfs at a 6 ft. reservoir surcharge, to elevation 173.5. However, the low point on the dike is at elevation 172.2, so that at the start of an overtopping the spillway capacity is 5,500 cfs. If the dike was raised to the elevation of the low point of the dam (177.3), the spillway capacity is computed to be about 18,500 cfs for this surcharge elevation. A spillway discharge curve is shown on Fig. 2, Sheet D-4, and computations are shown on Sheet D-5, Appendix D.

4. Spillway Capacity at Test Flood Elevation

The spillway capacity at test flood elevation is computed to be about 14,400 cfs at reservoir surcharge elevation 175.9.

5. Total Project Discharge at Test Flood Elevation

In addition to the spillway discharge, about 2,900 cfs would be discharged over the end of the dike at the right abutment, giving a total project discharge at test flood elevation of about 17,200 cfs at reservoir surcharge elevation 175.9.

c. Elevation (ft. above MSL)

1. Streambed at centerline of dam - 139.5
2. Maximum tailwater for 8,500 cfs - Not calculated
3. Upstream portal invert, diversion conduit - 144.5
4. Recreation pool - Not applicable
5. Full flood control pool - Not applicable
6. Spillway crest - 167.5
7. Design surcharge - 173.5
8. Top of dam - 177.3 to 180.3
9. Top of dike - 172.2 to 177.3
10. Test flood design surcharge - 175.9

d. Reservoir

1. Length of maximum pool - 2.04 miles
2. Length of recreation pool - Not applicable
3. Length of flood control pool - Not applicable

e. Storage (acre-feet)

1. Recreation pool - Not applicable
2. Flood control pool - Not applicable
3. Spillway crest pool El 167.5 - 1,050
4. Top of dam El 177.3 - 2,850
5. Top of dike El 172.2 - 1,750
6. Test flood pool El 175.9 - 2,500

f. Reservoir Surface (acres)

1. Recreation pool - Not applicable
2. Flood control pool - Not applicable
3. Spillway crest - 100
4. Test flood pool - 238
5. Top of dam - 258

g. Dam

1. Type - Zoned earth embankment
2. Length - 1,018 ft.
3. Height - Variable, 38 ft. to 41 ft.
4. Top width - Variable width on right abutment bench. 35 ft. right of spillway to right abutment bench. 20 ft. left of spillway.
5. Side slopes - 2½ to 1 upstream; 2 to 1 downstream
6. Zoning - Original dam: Impervious upstream and central zone, pervious downstream facing.
   New dam at breached section - Impervious upstream zone, random pervious central core, pervious downstream zone.
7. Impervious core - Original dam: Impervious upstream and central zone
   New dam at breached section: Impervious upstream zone
8. Cut-off - Cut-off trench at upstream toe, backfilled with impervious fill. Trench depth to bedrock or maximum of 10 ft. in earth.
9. Grout curtain - None
10. Other - 18 in. riprap on filter bedding on upstream face. Sodded downstream face.

Right (West) Abutment Dike

1. Type - Random pervious fill with upstream impervious blanket underlying riprap facing.
2. Length - 180 ft.
3. Height - 14 ft. maximum
4. Top width - 10 ft.
5. Side slopes - 2½ to 1 upstream; 2 to 1 downstream
6. Zoning - Impervious upstream zone tied into upstream cut-off trench, random pervious central zone, pervious downstream zone.
7. Impervious core - Impervious upstream zone
8. Cut-off - 5 ft. deep cut-off trench at upstream toe, backfilled with impervious material.
9. Grout curtain - None.
10. Others - 18 in. riprap on filter bedding on upstream face. Sodded downstream face.

h. Diversion and Regulating Tunnel - None

i. Spillway

1. Type - Ungated stepped-chute with riprapped trapezoidal stilling basin
2. Length of weir - 150 ft.
3. Crest elevation - 167.5 ft.
4. Gates - None
5. Upstream channel - 150 ft. wide channel, 100 ft. long from reservoir to ogee overflow.
6. Downstream channel - Converging chute from 150 ft. width to 90 ft. width in 60 ft. length. 90 ft. wide rectangular concrete chute 177 ft. long, stepped with 6 ft., 8 ft. and 16 ft. vertical drops.
7. General - Spillway flows directed into channelized river section through Social area of Woonsocket.

j. Regulating Outlets

1. Invert - 144.5
2. Size - 36 in. dia.
3. Description - Concrete pipe conduit
4. Control Mechanism - 24 in. dia. butterfly valves
5. Other - Two 16 in. dia. cast iron inlet pipes at elevations 160.5 and 153.0 lead into a wet well shaft where inflows are controlled by butterfly valves. A 20 in. dia. pipeline conveys these flows to a pumping station. There is a 6 in. dia. bypass valve on a tee in the 20 in. dia. pipe.

SECTION 2 - ENGINEERING DATA

2.1 Design

No data on the design of the original nineteenth century dam has been recovered and probably none exists. The 1968-69 rehabilitation of the remains of the original dam was designed by Metcalf & Eddy, Engineers (now Metcalf & Eddy, Inc.) of Boston, Massachusetts. Copies of drawings which are pertinent to considerations of dam safety are included in Appendix B. Design data is available on microfilm at the library of Metcalf & Eddy at 50 Staniford Street, Boston, where it has been reviewed by the inspection team.

2.2 Construction

No information relating to construction of the original dam has been found. The reconstructed dam was built in 1968-69 by contract under the supervision of the design engineers, but no construction records have been located.

2.3 Operation

The dam and reservoir are operated by personnel of the City of Woonsocket Department of Public Works, in conjunction with other water storage facilities in the city's water supply system. According to the Division Engineer, Harris Pond has only been utilized as a source of water occasionally for short periods since the dam was reconstructed. The 6 in. dia. bypass valve is opened when the reservoir storage level falls below normal and there is no discharge over the spillway. The pumps are tested annually.

2.4 Evaluation

a. Availability

The plans, specifications, boring logs and microfilm file of engineering data in the design engineer's library, supplemented by the visual observations of the inspection team, form the basis for the information presented in this report.

b. Adequacy

The lack of in-depth engineering data did not allow for a definitive review. Therefore, the adequacy of this dam could not be assessed from the standpoint of reviewing design and construction data, but is based primarily on visual inspection, past performance history and sound engineering judgment.

c. Validity

The validity of the engineering data acquired covering the dam and spillway structure is considered acceptable and is not challenged.

SECTION 3 - VISUAL INSPECTION

3.1 Findings

a. General

The visual inspection of Harris Pond Dam took place on 27 September 1978. At that time the reservoir was about at normal storage level, with a small discharge over the spillway crest. Both the dam and the supplementary dike at the right abutment were judged to be in good condition. There was no evidence of any major maintenance problems, but several minor items require attention (see Section 7.3).

b. Dam

The profile of the crest of the dam is not horizontal, but slopes each way from the spillway. The original design drawings show the dam crest as level at elevation 177.3. Record drawings, however, show that the top of the dam is at elevation 180.3 at the abutments of the spillway, and that it ramps down to the left to elevation 177.3 about opposite the outlet control house, and to the right to elevation 177.3 at the right abutment.

The crest of the main embankment to the right of the spillway has a good horizontal and vertical alignment. No cracks were observed in the crest pavement, which is in good condition. The alignment of the crest of the dam to the left of the spillway is also good, while the crest pavement is in generally good condition, except for a transverse crack across the pavement about 20 ft. to the left of the gatehouse. The crack is about ½ in. wide and 1 in. deep.

The riprap on the upstream slope of the dam is in excellent condition. There is some minor erosion of the embankment material at the intersections of both spillway training walls, where it appears that some of the rock riprap was removed by vandals (Appendix C, Photo No. 1).

The downstream slope of the dam is in excellent condition, with no evidence of bulges or movement (Appendix C, Photo No. 3). The downstream toe of the dam to the right of the spillway has a cobblestone facing for the bottom 3 ft., measured along the slope. These are rounded stones roughly 6 in. to 10 in. in diameter. The area immediately

below the toe of the dam for a width of 300 ft. is essentially flat, and the ground is wet where the cobblestone armor intersects the natural ground. However, no major springs or seeps in the embankment were observed. The seepage which is evident collects toward the right abutment along the intersection of the embankment and natural ground. An 8 in. dia. toe drain outlet pipe emerges from the toe of the dam near its intersection with the right abutment; the flow from this pipe was estimated at about 5 gpm.

At the toe of the dam near the left abutment, a small seep was noted on the downstream slope immediately to the left of the 36 in. conduit concrete outlet structure. This seepage was estimated to be of the order of 1 gpm. The 36 in. outlet pipe was not discharging at the time and was dry. Along the downstream toe of the dam, between the outlet structure and the spillway channel, the ground showed seeps totaling an estimated 20 to 30 gpm. Some surface drainage from the housing project, located downstream from the left abutment toe, also drains into the outlet channel, and some of the evident flow may originate from this source as well as from the seeps.

Considering the dam in its entirety, no major localized seeps were noted. Seepage flow was clear and was not carrying solids.

The top of the right abutment dike slopes from elevation 177.2 at the end of the dam to about elevation 172.1 at the end of the dike about 180 ft. away. The area downstream from the dike has been filled in and paved contiguous with the dike, and is now used for a parking lot for the mill building nearby. In the event of an encroachment on the reservoir freeboard above elevation 172, this saddle area would act as an auxiliary spillway for discharging flows from the reservoir. Any such discharges would flow back into the Mill River upstream of the Mill River Conduit headwall.

Riprap on the upstream slope of the dike and the crest paving on the dike is in good condition, except for some widely spaced shrinkage cracks in the pavement (Appendix C, Photo No. 4). Trespassers have displaced some riprap locally to form stepping stones around the fence. No seepage was observed along those parts of the downstream toe of the dike which were not paved over by the parking lot surfacing.

c. Appurtenant Structures

The spillway is founded on an earth knoll with undetermined overburden depth to bedrock. The spillway inlet channel is lined with a 3 ft. thick impervious earth blanket protected by a riprap surfacing. The downstream channel floor slabs are underlain with a gravel blanket and a sewer pipe drainage system. With about 1 in. of flow over the spillway crest and in the spillway channel, no seepage flow from the outlets of the underdrainage system could be observed (Appendix C, Photo No. 6).

The condition of the concrete at the spillway bridge, walls and floor, where not covered by flow, appeared to be excellent (Appendix C, Photo No. 5). For higher spillway flows, the stilling basin will perform as a plunge pool and some displacement of the riprap can be expected. The riprap appeared to be fairly well distributed and uniformly covering the basin surface.

The outlet works control shaft and gatehouse, where visible, appeared in excellent condition. Except for the possible seepage around the outside of the low-level outlet structure, as noted previously, the outlet conduit terminal appeared stable (Appendix C, Photo No. 7).

d. Reservoir Area

The banks upstream from the dam on both abutments slope gradually and appear stable against sloughing into the reservoir. The reservoir is crossed at several points by the Penn Central R.R. and an abandoned rail bed, and by the Farm Street roadway. These rail and road embankments divide the reservoir into separated pondages, interconnected by culverts at varying invert levels. Thus, below normal storage level, not all ponds are contiguous with the main reservoir. In the surcharge storage space, the levels in the auxiliary ponds and in the main reservoir would equalize.

e. Downstream channel

The Mill River below Harris Pond Dam has been straightened and channelized for about 0.6 miles (Appendix C, Photo Nos. 2, 9 & 10), after which it flows through a 12 ft. high by 42 ft. wide twin barrel conduit, to empty into the Blackstone River (Appendix C, Photo Nos. 11 & 12). The channelized portion of the river valley is diked above the conduit so as to prevent floods from overflowing onto the Social area of Woonsocket. The tops of the dikes are at about elevation 144, or about 8 ft. higher than the streambed below the dam.

Privilege Street and School Street bridges cross the river channel upstream from the flood control conduit. These bridges would provide some restrictions to large flows down the Mill River channel. The major restriction to such flows, however, would be that of backwater owing to the control capacities through the flood control conduit, dictated initially by the flow stage in the Blackstone River at the conduit outlet.

3.2 Evaluation

The visual inspection of the dam adequately revealed key characteristics as they may relate to its stability and integrity, permitting an assessment to be made of those features affecting the safety of the structure. The Harris Pond Dam and appurtenant works are judged to be in generally good condition.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 Procedures

The Harris Pond Dam is operated by personnel of the Woonsocket Department of Public Works. According to the Division Engineer, Harris Pond has only been used as a water source occasionally for short periods. The 6 in. dia. bypass valve is opened to provide some downstream flow whenever the reservoir storage level falls below normal. The pumps are tested annually. No documented operating procedures have been prepared.

4.2 Maintenance of Dam and Dike

Maintenance is carried out as required by city personnel. This consists principally of the periodic cutting of brush and other growth on the dam embankment, and repair of riprap and fences damaged by trespassers.

4.3 Maintenance of Operating Facilities

All manually operated valves are said to be serviceable and inspected regularly, except for the upstream 24 in. dia. butterfly valve, which is jammed in the open position. The gatehouse is kept locked and secure, but attempts have been made by unauthorized persons to break down the door.

4.4 Warning System

There is no formal warning system at this dam.

4.5 Evaluation

Operational procedures should be formalized and put into writing. The level of effort put into routine maintenance requires increasing slightly. Operating facilities should be repaired as necessary and a flood warning plan should be developed and implemented.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 Evaluation of Features

a. General

The drainage area above Harris Pond covers about 32.5 square miles. On the basis of envelope curve values given in the March 1978 Preliminary Guidance for Estimating Maximum Probable Discharges (NED), a range of CSM values between 600 and 1,500 (flat to rolling terrain) are suggested. If a CSM value of 1,000 is assumed, the resultant peak inflow into Harris Pond is about 33,000 cfs.

As noted in Section 1.3a, however, the Mill River and tributary streams encompass a number of sub-areas, each of which has differing configurations, slopes, lag times, impoundments and runoff characteristics. Since use of the guidance curves does not take into acount these variations, and since the flood peak of the hydrograph is an important consideration in this case, it was decided to analyze the drainage basin by sub-areas. In this way the different runoff influences and retardations caused by the various reservoir impoundments, and the spillway outflow characteristics at these installations, are taken into consideration.

In 1960, prior to the reconstruction of Harris Pond Dam, the New England Division, Corps of Engineers, designed and constructed the Mill River flood control conduit below the dam. The design capacity of this conduit is indicated as being 8,500 cfs, equivalent to 262 CSM (not considering inflows from Peters River).

The rehabilitation of Harris Pond Dam was designed in 1968 by Metcalf & Eddy, Engineers, of Boston, MA. Microfilm files of design data have been reviewed, but no computations covering the hydrology of the drainage basin have been found. It was determined that, when the capacity of the new spillway was being considered, the designers decided to adopt the 8,500 cfs design capacity used for the Mill River conduit as appropriate for spillway design. Because such criteria as rainfall-duration, antecedent saturation conditions and infiltration losses on the area, lag times, etc. were apparently not considered in design studies, comparisons between the design procedure and those utilized for this Phase I Report cannot be made.

Harris Pond is basically a low surcharge-high spillage project. The dam is a rehabilitated nineteenth century earth embankment with new spillway and outlet structures.

b. Design Data

No design data, other than the spillway design capacity, was recovered.

c. Experience Data

As noted in Section 1.3b, the maximum recorded flood, which resulted in the breaching of the spillway at the original dam, occurred in August 1955. Other high flow runoffs occurred in 1968, but records of runoff magnitude were not retrieved.

d. Visual Observations

No evidence was noted to indicate possible high flows through the reservoir and in the downstream channel since the reconstruction of the dam and rechannelization downstream.

e. Test Flood Analysis

1. Drainage Areas

The 32.5 square mile basin drainage area above Harris Pond Dam was divided into 8 sub-areas for the hydrologic analysis. A flood hydrograph was prepared for each sub-area and flood routings were conducted where flows passed through reservoirs sited on the streams. These sub-areas are tabulated on Sheet D-2, noting location, size of drainage area, water course length and river slope.

2. Reservoir Areas and Capacities

The Harris Pond Reservoir at normal storage level impounds about 1,050 acre-ft. For determining reservoir surcharge storage capacity, planimetered areas were taken from contours delineated on the USGS 2,000 ft. per in. quadrangle sheets. Area-capacity curves for Harris Pond are shown on Figure 3, Sheet D-6, Appendix D. Computations for the area-capacities are shown on Sheet D-7.

Storage capacities of impoundments above Hopedale Reservoir Dam (Freedom Street Dam) are shown on Sheet D-8.

For determining surcharge storages at the upstream reservoirs for use in flood routings, areas were similarly planimetered and storages computed. North Pond areas and capacities are shown on Sheets D-9 and D-11, Appendix D; Hopedale Reservoir areas and capacities are shown on Sheets D-12 and D-13; Lake Hiawatha areas and capacities are shown on Sheet D-15.

3. Outflow discharge capacities

For use in the flood routings of the inflows through the various impoundments, discharges were computed through the spillways and over the tops of the dams on the several installations where outflows would be considerably retarded. These are shown for North Pond on Sheets D-10 and D-11; for Hopedale Reservoir on Sheets D-13 and D-14; and for Lake Hiawatha on Sheet D-15. Spillway and dam overtopping capacities for Harris Pond Dam are shown on Sheet D-4. The discharge and surcharge storage capacities for flood routing inputs are summarized on Sheet D-16.

4. Test Flood

Harris Pond Dam is about 40 ft. high and impounds about 2,850 acre-ft. to top of dam. As stated in Section 1.2, it is therefore classified as intermediate in size. Because of downstream conditions, the hazard potential is classified as high. The Recommended Guidelines for Safety Inspection of Dams requires that for hydraulic evaluation the dam adequacy be tested for a full PMF.

5. Precipitation Data

Precipitation data was obtained from Hydrometeorological Report No. 33, which for the Woonsocket area in Rhode Island approximates 23.5 in. of 6 hour point rainfall over a 10 square mile area. This value was reduced by 12 percent to apply to a 32.5 square mile total area, and by an additional 17 percent to conform to the area fit reduction criteria. The 6 hour rainfall was distributed into ½ hour incremental periods as suggested in COE Publication EC 1110-2-1411. Infiltration losses of 1 in. during the first hour and 0.2 in. during each succeeding hour were assumed. The net rainfall excesses for developing the runoff hydrographs are shown on Sheet D-17, Appendix D.

6. Drainage Basin Criteria

To evaluate the sub-drainage basin characteristics for the lag and transport times required to develop the sub-basin hydrographs and upstream reservoir outflow patterns, a stream profile of the various water courses and pondages was prepared from the USGS quadrangle sheets. This profile is shown on Figure 4, Sheet D-18. The incremental stream lengths for each sub-drainage basin were then evaluated for time of concentration, lag time and resulting flow velocity. The resulting values are recorded on Sheets D-19 and D-20, Appendix D. Times of concentration and lag times were selected so as to produce a weighted average equivalent flow velocity within the various sub-basin streams of about 0.75 ft. per sec., and a transport velocity between sub-basins of about 1.2 ft. per sec.

7. Selected Unitgraphs

The unitgraph utilized for developing the various sub-basin inflow hydrographs is the curvilinear adaptation of a triangular unitgraph, shaped as described in Design of Small Dams. These unitgraphs for the variously adopted time-to-peak values selected for the differing sub-basins are shown on Sheets D-21 and D-22.

As a check on the validity of the time-of-concentration velocity values selected on Sheets D-19 and D-20, different cross sections and the average slope along the reach of the Mill River upstream from Forge Pond were measured from the USGS Quadrangle sheets, and a composite representative cross section was assumed. Based on an n value of 0.10, a stage-velocity and stage-discharge relationship was computed (see Sheet D-23). The computed velocity values varied from about 1 to 2 ft. per second, depending on river discharge. The values used for determining lag and time-of-concentration, being lower than this, thus appear to have been selected on the conservative side.

8. Runoff Hydrographs and Flood Routings

Runoff hydrographs were prepared for each of the sub-areas selected, and were combined to form the inflow hydrograph into Harris Pond reservoir, after the appropriate routings through North Pond, Hopedale

reservoir and Lake Hiawatha. This combination of sub-hydrographs is plotted on Fig. 5, Sheet D-24, to represent the PMF inflow for routing through Harris Pond reservoir and spillway. The maximum PMF inflow is 17,500 cfs. The resulting flood routing through Harris Pond reservoir shown on Figure 6, Sheet D-25, indicates a maximum outflow of 17,200 cfs at surcharge storage head elevation 175.9. Of this discharge, 14,400 cfs would be released through the spillway and 2,900 cfs would spill over the right end of the dike and along the mill parking lot (see plan and profile on Figure 1, Sheet D-1). For this flood, approximately 3,000 acre-ft. of the runoff would be spilled over the dike and right abutment during a duration of 20 hrs.

In the event that the dike and reservoir rim to the right were to be raised to forestall flows around the right abutment, the reservoir would rise to about elevation 176.8, with the total outflow of about 17,200 cfs spilling entirely through the spillway.

The 0.5 PMF hydrograph shows a peak inflow of about 8,750 cfs, approximately that of the Standard Project Flood used for the design of the dam and spillway. This hydrograph is shown on Fig. 7, Sheet D-26. The reservoir surcharge level resulting from a routing of this flood would be about elevation 173.5, which is about 3.8 ft. below the low point on the dam crest. However, the end of the right dike and reservoir rim to the right would be overtopped to a maximum of about 1.3 ft.

9. Downstream Channel Capacity

As noted in Section 1.2d and 3.1e, the Mill River channel downstream from Harris Pond Dam has been channelized and diked, and a floodway conduit has been constructed under the Social area of Woonsocket. The waterway and conduit were designed for a Standard Project Flood capacity of 8,500 cfs. Computed on Sheet D-27 and plotted on Figure 8, Sheet D-28, are capacities of the conduit for differing flood stage conditions in the Blackstone River, and their backwater effect on headwater level in the channel upstream. It will be noted that an overtopping of the dikes above the conduit would occur whenever outflows from Harris Pond Dam exceeded about

11,000 to 13,000 cfs, depending upon the stage of flow in the Blackstone River for these flows. It is thus evident that, although Harris Pond Dam could accommodate a PMF inflow without being overtopped if the dike was raised to the elevation of the dam, the downstream channel capacity is insufficient to avoid resultant flooding of the Social area of Woonsocket.

10. Spillway Adequacy

While the spillway crest has sufficient capacity to discharge up to 18,500 cfs with reservoir level to the low point of the dam's crest, it appears that the spillway chute would accommodate less than the design 8,500 cfs. outflow before its walls would be overtopped.

In the stepped spillway chute, much of the energy generated in the total drop from spillway crest to downstream river level will be dissipated at each successive floor level drop in the chute. Subcritical flows downstream from the jet trajectory at each drop will prevail, passing through critical flow only at the edges of the drops. Illustrated on Sheet D-29 is a graphic representation of flow conditions as they are expected to prevail, with computations of flow depths for various discharges through the chute. It will be noted that, disregarding swell owing to air entrainment and high turbulences from boil and wave action caused by dissipation below each free falling jet, the walls would be expected to be overtopped for discharges in excess of about 8,000 cfs.

A condition which further aggravates flows in the chute and would threaten an overtopping of the side walls is the sharp convergence of the chute upstream from the first step below the crest. This convergence will cause impingement and high waves, which will ride up and overtop the walls. Furthermore, the absence of aeration below each step will result in subatmospheric pressure under the overflowing jets, causing make-and-break action to further disturb the flow in the chute and threaten an overtopping of the walls. Thus, although the dam itself would not be overtopped from inflows of a magnitude up to the full PMF test flood, it is possible that the safety of the dam could be threatened by a washout of the spillway for inflows of even less than a 0.5 PMF.

f. Dam Failure Analysis

Although Harris Dam proper would not be overtopped for a PMF test flood, even with a flood inflow of less than a 0.5 PMF event, an overtopping of the right end of the right dike, and of the reservoir rim beyond, would occur. As illustrated in Section 5f, failure of the spillway at a flood event of about 0.5 PMF could threaten a breach in the main dam.

As noted in Para. 5.1e(6), any outflow from Harris Pond reservoir in excess of about 11,000 to 13,000 cfs, either from normal releases without failure of the structures or from a breach failure, will overtop the floodway channel and result in flooding of the Social area of Woonsocket.

Though an overtopping of the dam at its maximum section near the center of the dam, or at its left end where the 1955 breach occurred, is not indicated for floods up to a full PMF, a structural failure owing to piping or sloughing could occur. A breach from that cause would be similar to that from an overtopping and the "rule of thumb" criteria suggested in the NED March 1978 Guidance Report would be applicable, except that the reservoir at such a time would not be higher than the surcharge resulting from a PMF inflow. A reservoir water surface of about elevation 175 was therefore assumed for computing the breach outflow. Computations on Sheet D-30 show an outflow of the order of 24,000 cfs.

The waterway under the Privilege Street bridge about 700 ft. below the dam will form a constriction for high outflows from Harris Pond reservoir. Computed on Sheet D-31 and plotted on Sheet D-32 are stage-discharges upstream from the bridge, assuming the river bottom at the bridge to be at elevation 136.

For a breach outflow from the dam approaching 24,000 cfs. it is possible for the water level upstream from the bridge to rise to between elevation 157 and 163, depending on whether the bridge would remain in place under a flood surge onslaught. Within the area of inundation are a five story, high-rise apartment building whose base level is at about elevation 148, several warehouse buildings and other commercial establishments.

The rechannelized river floodway dikes and flood control conduit below the dam have their tops at elevation 142. Additionally, the valley storage in the reach from Harris Pond Dam to the upstream end of the conduit between the levee dikes is not large. Thus, a large flood surge from Harris Pond reservoir would prevail through the length of the rechannelized floodway, and would overtop the confining dikes when discharges exceeded about 10,000 to 12,000 cfs. This would result in flooding of the Social district of Woonsocket to a depth of at least 10 ft. This is a densely developed area of individual homes, high-rise apartment and office buildings, and commercial establishments.

Delineated on Sheet D-33, in Appendix D, are the areas which could be flooded by a breach failure of the dam (Quad sheet graphic).

SECTION 6 - STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

a. Visual Observations

The field investigations of the earth embankment revealed no significant displacements or distress which would warrant the preparation of slope stability computations based on assumed soil properties and engineering factors. Data available on the engineering characteristics of the dam is limited to the new embankment material placed during reconstruction of the dam in 1968. The reconstruction incorporated the original embankment which was reported to consist of fine to coarse sand with gravel. The modifications utilized a sloping impervious base on the upstream side, consisting of silty sand with gravel and with pervious filter layers and riprap. The downstream slope was covered with pervious fill and a toe drainage collection system was added.

b. Design and Construction Data

Design plans for the 1968 reconstruction were reviewed. However, since shear strength data of the original embankment material and foundation were not available, a detailed stability analysis was not deemed worthwhile. The design of the reconstructed elements appear generally to be consistent with good earth dam embankment design practice. However, the use of a partial cutoff to bedrock under the upstream toe and the observed seepage indicate that periodic inspections are necessary during periods of high reservoir level, and at least once a year, to monitor the quantity and clarity of seepage from below the embankment.

The design drawings also indicate a minimum dimension of 3 ft. of impervious material at the junction of the cut-off trench and impervious upstream section. This value is considered to be less than that used in present day design practice. The actual "as built" minimum thickness is not known, however.

c. Operating Records

There are no formal operating records for this dam.

d. Post Construction Changes

The reconstruction of the dam accomplished in 1968 would not adversely affect the stability. The results of the field inspection and a check of the available records produced no evidence of other changes which might impair stability.

e. Seismic Stability

The dam is located in Seismic Zone No. 2 and, in accordance with recommended Phase I guidelines, does not warrant seismic analysis.

SECTION 7 - ASSESSMENT, RECOMMENDATIONS & REMEDIAL MEASURES

7.1 Dam Assessment

a. Condition

On the basis of the Phase I visual examination, the Harris Pond Dam appears to be in good condition and functioning adequately. The spillway capacity, however, is not adequate to pass the test flood outflow. This study indicates that further investigations are required and that additional routine maintenance is also needed.

There is some seepage along the downstream toe of the dam, particularly in the area between the outlet structure and the spillway. One of the 24 in. dia. outlet control valves is unserviceable. There is some minor erosion of the upstream embankment slope adjacent to the spillway walls. Brush and light tree growth are established in the outlet channel, and there is some light brush on the upstream slope in the vicinity of the spillway.

b. Adequacy of Information

The information recovered is considered adequate for the purpose of making an assessment of the performance of the dam.

c. Urgency

The recommendations and remedial measures enumerated below should be implemented by the owner within one year after receipt of the Phase I Inspection Report.

d. Need for Additional Investigation

Additional investigations are required as recommended in Para. 7.2.

7.2 Recommendations

It is recommended that the owner should retain the services of a competent registered professional engineer to make investigations and studies of the following, and if proved necessary, to design appropriate remedial works:

1. Make a thorough study of the hydrology of the drainage basin. Review the situation regarding the potential overtopping of the dike and right abutment, and determine whether the dike should be raised and extended.

2. Review the spillway flow conditions below the crest of the dam through the converging section and through the chute reach. Study the feasibility of providing aeration below the overflow jets at each drop along the chute.

7.3 Remedial Measures

a. Operating and Maintenance Procedures

1. Minor erosion of the upstream embankment slope adjacent to both the right and left spillway walls should be repaired. Riprap displaced by trespassers in the vicinity of the fences should be replaced.

2. Brush growth on the upstream slope near the crest of the embankment right of the spillway should be removed and controlled on a regular basis.

3. Brush and light tree growth in the outlet channel should be removed.

4. Seepage along the downstream toe of the embankment and toe drain discharges should be monitored monthly during periods of high reservoir level and at least once a year, for changes in seepage volume and turbidity.

5. The unserviceable 24 in. dia. outlet valve should be repaired.

6. A formal surveillance and flood warning plan should be developed. An operational procedure to be followed in the event of an emergency should also be adopted.

7. Procedures for a biennial periodic technical inspection of the dam and appurtenant works should be instituted.

7.4 Alternatives

There are no practical alternatives to the above recommendations.

APPENDIX A

VISUAL INSPECTION CHECKLIST

VISUAL INSPECTION
PHASE I

Identification No. RI 03901 Name of Dam: Harris Pond Dam

Date of Inspection: 27 September 1978

Weather: Sunny, clear Temperature: $70^{o}F \pm$

Pool Elevation at Time of Inspection: 167.5 MSL

Tailwater Elevation at Time of Inspection: 136 $\pm$ MSL

INSPECTION PERSONNEL

Peter B. Dyson	Louis Berger & Associates, Inc.	Project Manager
Carl J. Hoffman	Louis Berger & Associates, Inc.	Hydraulics, Structures
Thomas C. Chapter	Louis Berger & Associates, Inc.	Hydrology, Soils
William S. Zoino	Goldberg Zoino Dunnicliff & Assoc., Inc.	Soils

OWNER'S REPRESENTATIVES

Hedley V. Patterson	City of Woonsocket	Division Engineer
Norman Desjardins	City of Woonsocket	Plant Maintenance Mechanic

VISUAL INSPECTION CHECKLIST

Identification No: RI 03901 | Name of Dam: Harris Pond Dam | Sheet 1

VISUAL EXAMINATION OF	OBSERVATIONS AND REMARKS
EMBANKMENT Vertical alignment and movement	No movement evident.
Horizontal alignment and movement	No movement evident.
Unusual movement or cracking at or near the toe	None evident.
Surface cracks	Minor sporadic cracks in asphalt pavement on crest; pavement generally good.
Animal burrows and tree growth	No burrows noted. No trees, some light brush.
Sloughing or erosion of slopes	None observed. Minor erosion adjacent to spillway walls.
Riprap slope protection	Crushed rock in generally good condition; some displacement due to trespassers climbing around fences.

VISUAL INSPECTION CHECKLIST

Identification No: RI 03901 Name of Dam: Harris Pond Dam Sheet 2

VISUAL EXAMINATION OF	OBSERVATIONS AND REMARKS
Seepage	Seepage noted at both abutments, perhaps totaling 50-75 gpm.
Piping or boils	None observed.
Junction of embankment and abutment, spillway and dam	Minor erosion of upstream slope near left spillway training wall. Vandalism of fences.
Foundation drainage	Toe drains functioning and discharging 3-4 gpm.
<u>OUTLET WORKS</u> Approach channel	N/A
Outlet conduit concrete surfaces	Tower concrete in good condition.
Intake structure	Two 16" Ø, one 36" Ø C.I. pipes. None visible.
Outlet structure	36" Ø C.I. pipe with R.C. headwall in good condition.

VISUAL INSPECTION CHECKLIST

Identification No: RI 03901 Name of Dam: Harris Pond Dam Sheet 3

VISUAL EXAMINATION OF	OBSERVATIONS AND REMARKS
Outlet channel	Overgrown with brush and weeds.
Drawdown facilities	Two 24" Ø butterfly valves, one broken in open position, installed in 36" Ø outlet pipe.
<u>SPILLWAY STRUCTURES</u> Concrete weir	In good condition.
Approach channel	Sloped entrance retaining walls in good condition.
Discharge channel	Three-stepped chute; concrete in good condition.
Stilling basin	Riprap lined plunge basin; riprap fairly well distributed.
Bridge and piers	P.S.C. box beam bridge 15' wide over weir.
Control gates and operating machinery	None.

VISUAL INSPECTION CHECKLIST

Identification No: RI 03901 | Name of Dam: Harris Pond Dam | Sheet 4

VISUAL EXAMINATION OF	OBSERVATIONS AND REMARKS
INSTRUMENTATION Headwater and tailwater gages	Tailwater gage unserviceable.
Embankment instrumentation	None.
Other instrumentation	None.
RESERVOIR Shoreline	Gently sloping, wooded, apparently stable.
Sedimentation	None observed.
Upstream hazard areas in event of backflooding	Houses close to shoreline on east side. Light industrial developments close to shoreline on west side.
Alterations to watershed affecting runoff	No recent alterations noted.

VISUAL INSPECTION CHECKLIST

Identification No: RI 03901 | Name of Dam: Harris Pond Dam | Sheet 5

VISUAL EXAMINATION OF	OBSERVATIONS AND REMARKS
<u>DOWNSTREAM CHANNEL</u> Constraints on operation of dam	Privilege Street Bridge across discharge channel 80 ft. span, 11.3 ft. vertical clearance. Mill River conduit to Blackstone River below dam.
Valley section	Channelized river section 50 ft.± wide with 2:1 side slopes. Wide valley above river section.
Slopes	Flat, some areas grass and trees, other areas developed.
Approx. No. of homes/population	Large 5 story apartment building at foot of dam. Businesses & homes on Privilege Street. Businesses and homes throughout area between dam and Blackstone River.
<u>OPERATION & MAINTENANCE FEATURES</u> Reservoir regulation plan, normal conditions	No formal plan. If discharge over weir ceases, 6" Ø sump drain valve opened.
Reservoir regulation plan, emergency conditions	No formal plan. No emergency has occurred since reconstruction of dam.

VISUAL INSPECTION CHECKLIST

Identification No: RI 03901 Name of Dam: Harris Pond Dam Sheet 6

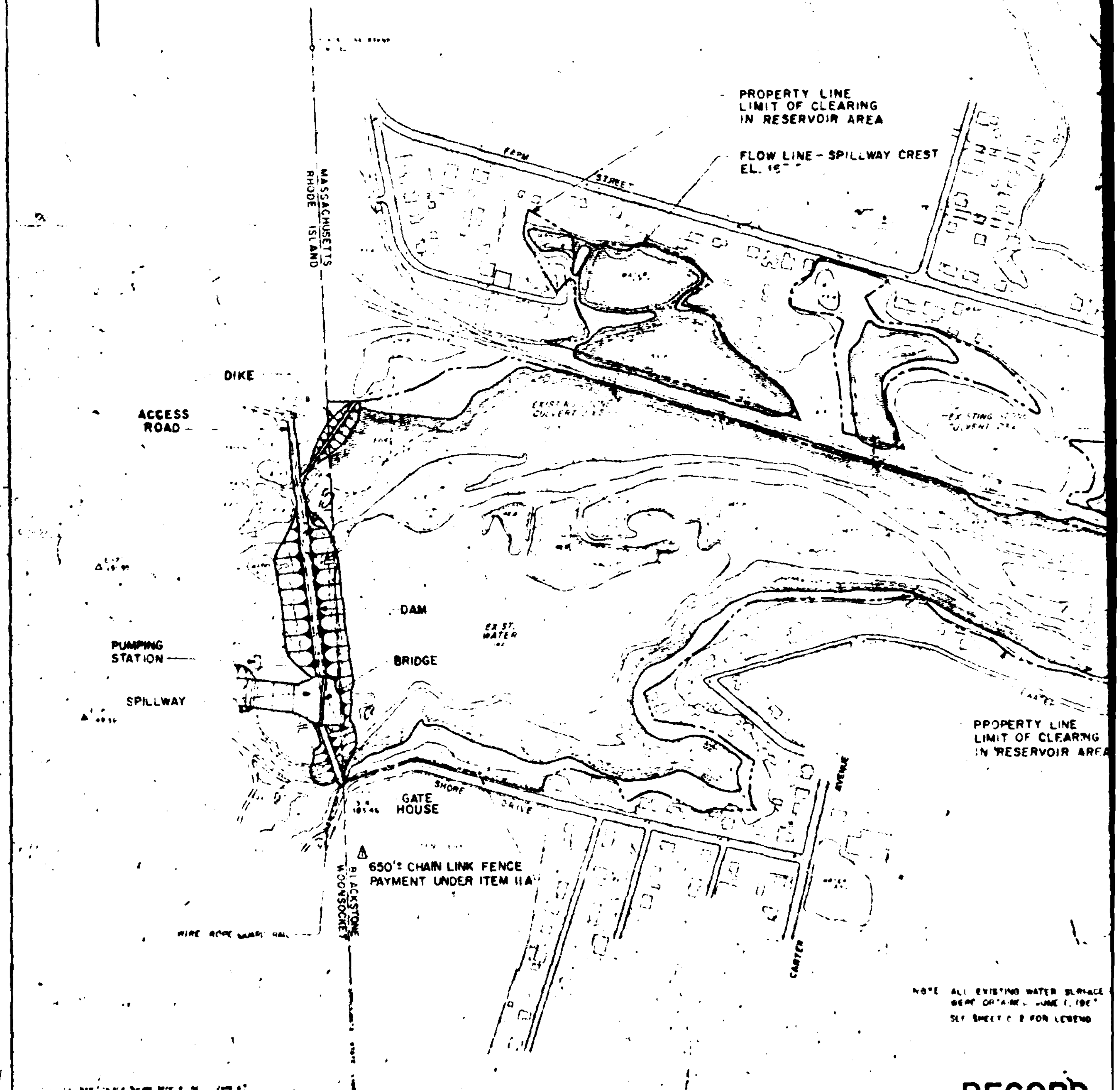
VISUAL EXAMINATION OF	OBSERVATIONS AND REMARKS
Maintenance features	No routine maintenance being performed.

APPENDIX B

PLANS & RECORDS

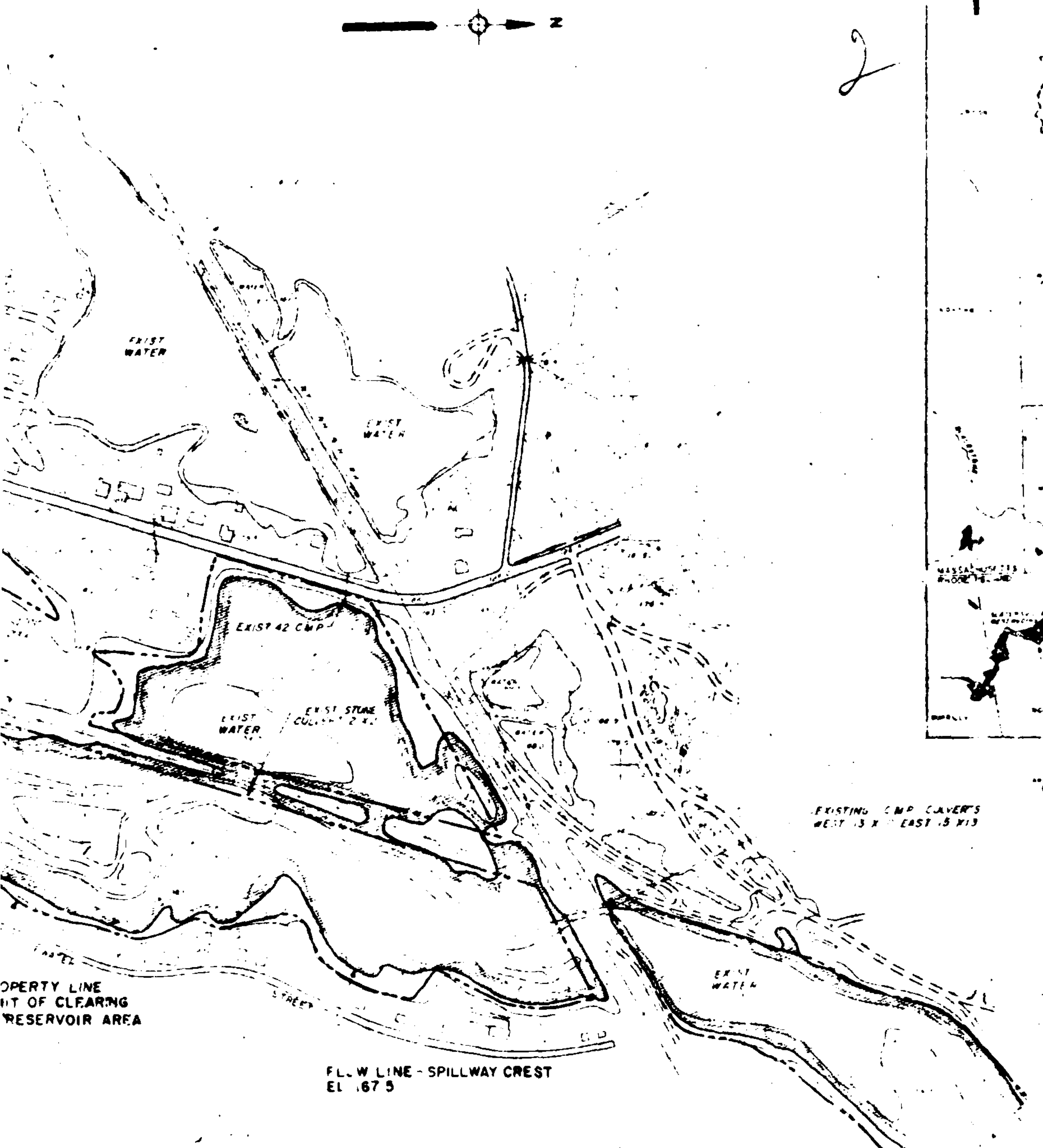
Drawings pertinent to dam safety were selected from files of Woonsocket Department of Public Works:

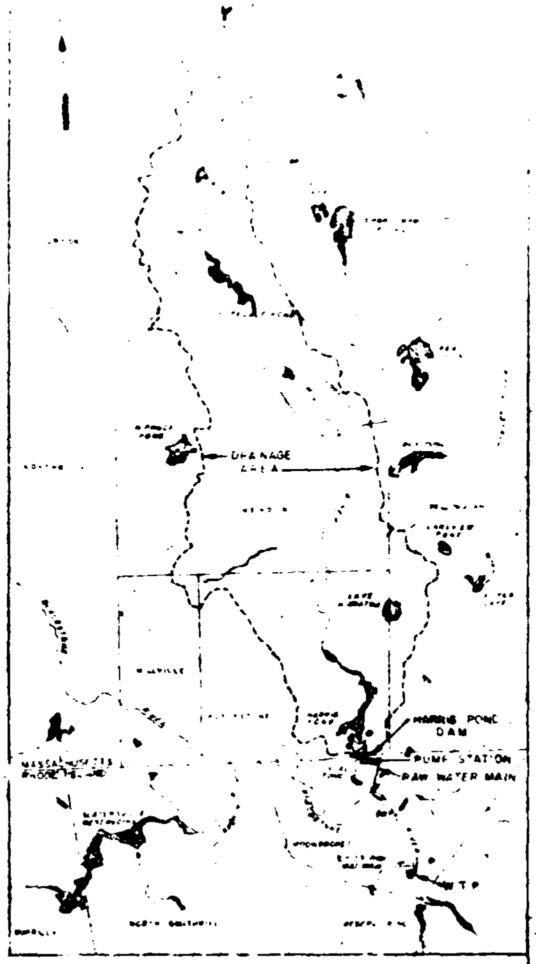
"Harris Pond Dam Rehabilitation, Pumping Station,
Raw Water Main & Appurtenant Work,
Contract 1967-1, HUD Project No. WS-1-40-0008."



PROPERTY LINE
LIMIT OF CLEARING
IN RESERVOIR AREA
FLOW LINE - SPILLWAY CREST
FARM
STREET
MASSACHUSETTS
RHODE ISLAND
DIKE
ACCESS
ROAD
DAM
BRIDGE
PUMPING
STATION
SPILLWAY
GATE
HOUSE
SHORE
DRIVE
650'± CHAIN LINK FENCE
PAYMENT UNDER ITEM IIA
BLACKSTONE
WOONSOCKET
AVENUE
CARTER
PROPERTY LINE
LIMIT OF CLEARING
IN RESERVOIR AREA

RECORD





VICINITY MAP

ECORD DRAWING

HUD PROJECT NO.

2	2-3-70	REB		RECORD DRAWING
REVISION		MADE BY	CHECKED BY	DESCRIPTION

SCALE

APPROVED

FOR METCALF & EDDY

JOHN S. BETHEL, JR.

REGISTERED

CITY OF WOONSOCKET, RHODE ISLAND

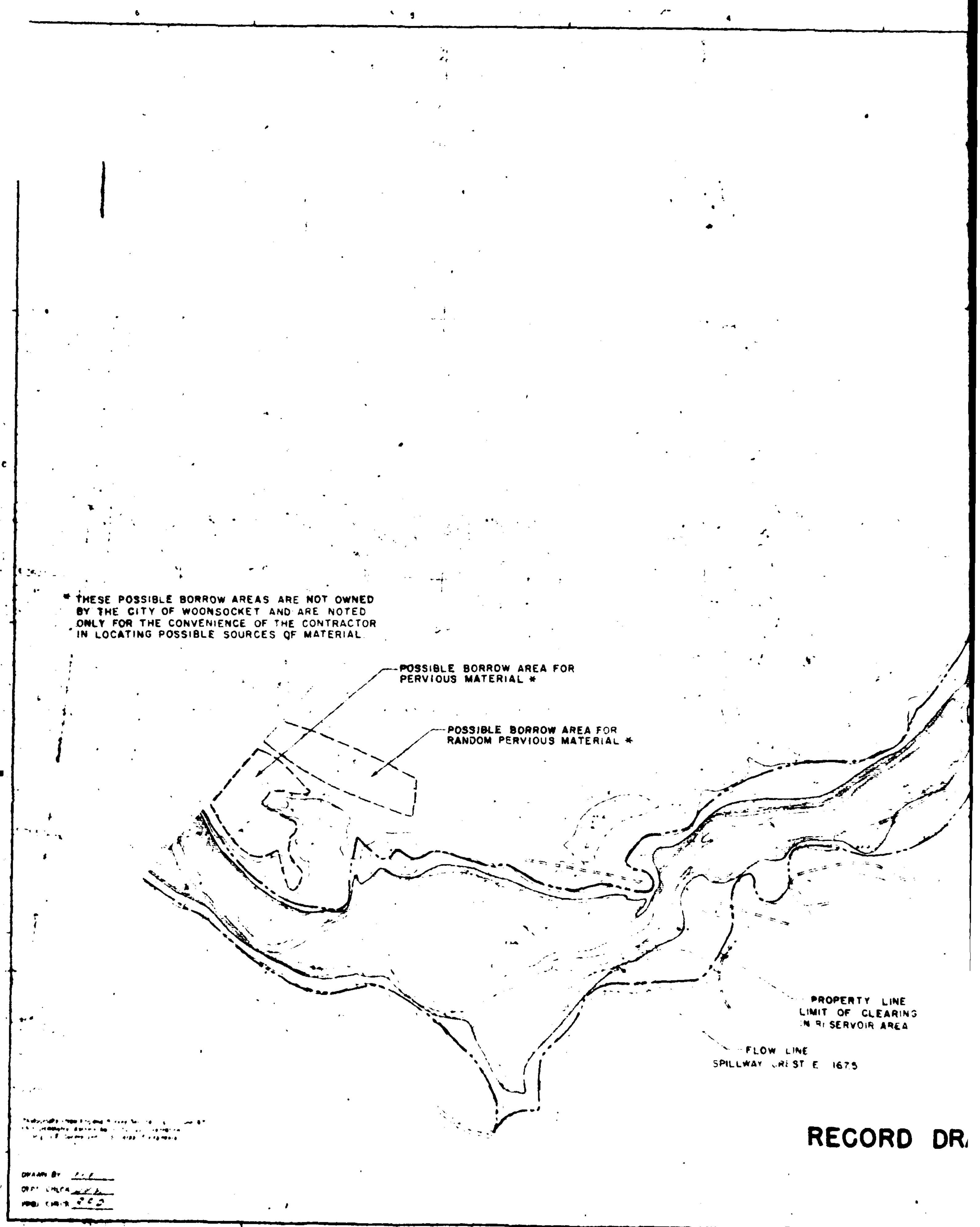
RESERVOIR SITE PLAN

METCALF & EDDY | ENGINEERS

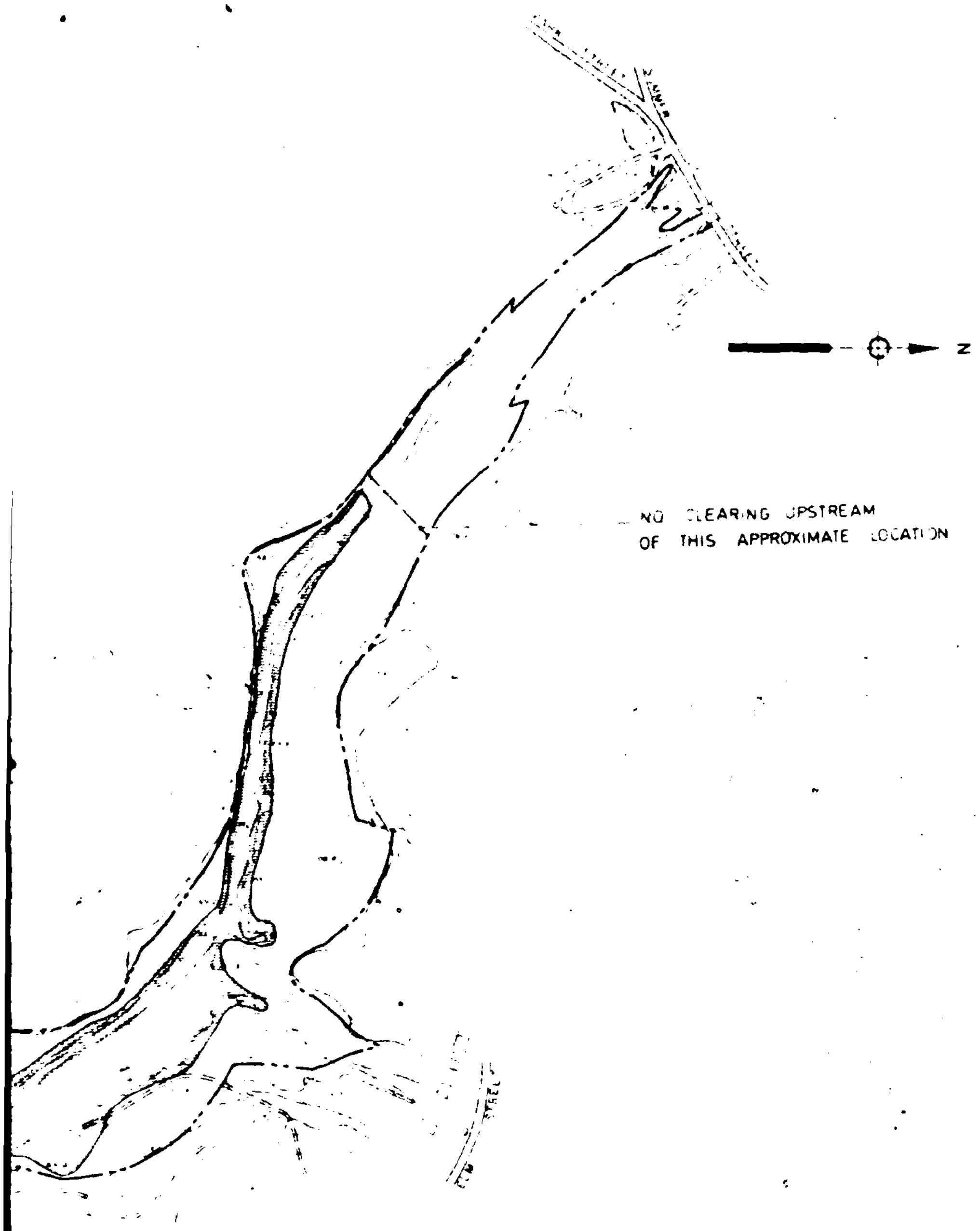
22-1688

CONTRACT

SHEET


* THESE POSSIBLE BORROW AREAS ARE NOT OWNED BY THE CITY OF WOONSOCKET AND ARE NOTED ONLY FOR THE CONVENIENCE OF THE CONTRACTOR IN LOCATING POSSIBLE SOURCES OF MATERIAL
POSSIBLE BORROW AREA FOR PERVIOUS MATERIAL *
POSSIBLE BORROW AREA FOR RANDOM PERVIOUS MATERIAL *
PROPERTY LINE
LIMIT OF CLEARING
IN RESERVOIR AREA
FLOW LINE
SPILLWAY CREST E 1675
RECORD DR
DRAWN BY

2



LEGEND

SMH	EXISTING SEWER MANHOLE
DMH	EXISTING DRAIN MANHOLE
TMH	EXISTING TELEPHONE MANHOLE
GG	EXISTING GAS GATE
WG	EXISTING WATER GATE
WSO	EXISTING WATER SHUT OFF VALVE
HYD	EXISTING HYDRANT
	PROPERTY LINE - LIMIT OF CLEARING
	FLOW LINE - SPILLWAY CREST
	EXISTING FLOW LINE
	SWAMP
	WOODS OR BRUSH
	EXISTING CONTOUR
	EXISTING SPOT ELEVATION
	PROPOSED EMBANKMENT
EL 182.0	SPOT ELEVATION
	BORINGS

GENERAL NOTES

1. ELEVATIONS IN FEET REFER TO MEAN SEA LEVEL U.S.G.S.
2. GRID SYSTEM CONFORMS TO THAT USED BY THE U.S. CORPS OF ENGINEERS LOWER BLACKSTONE FLOOD CONTROL PR...
3. FOR STRUCTURAL GENERAL STANDARDS SEE SHEET S 9
4. EXCAVATION FOR EMBANKMENTS TO BE PAID FOR AS STRIPPING UNLESS OTHERWISE SPECIFIED
5. FOR BORING LOGS SEE SPECIFICATIONS

PERTY LINE
OF CLEARING
ERVOIR AREA

675

ECORD DRAWING

HUD PROJECT NO

RECORD DRAWING

SCALE

APPROVED

JOHN S

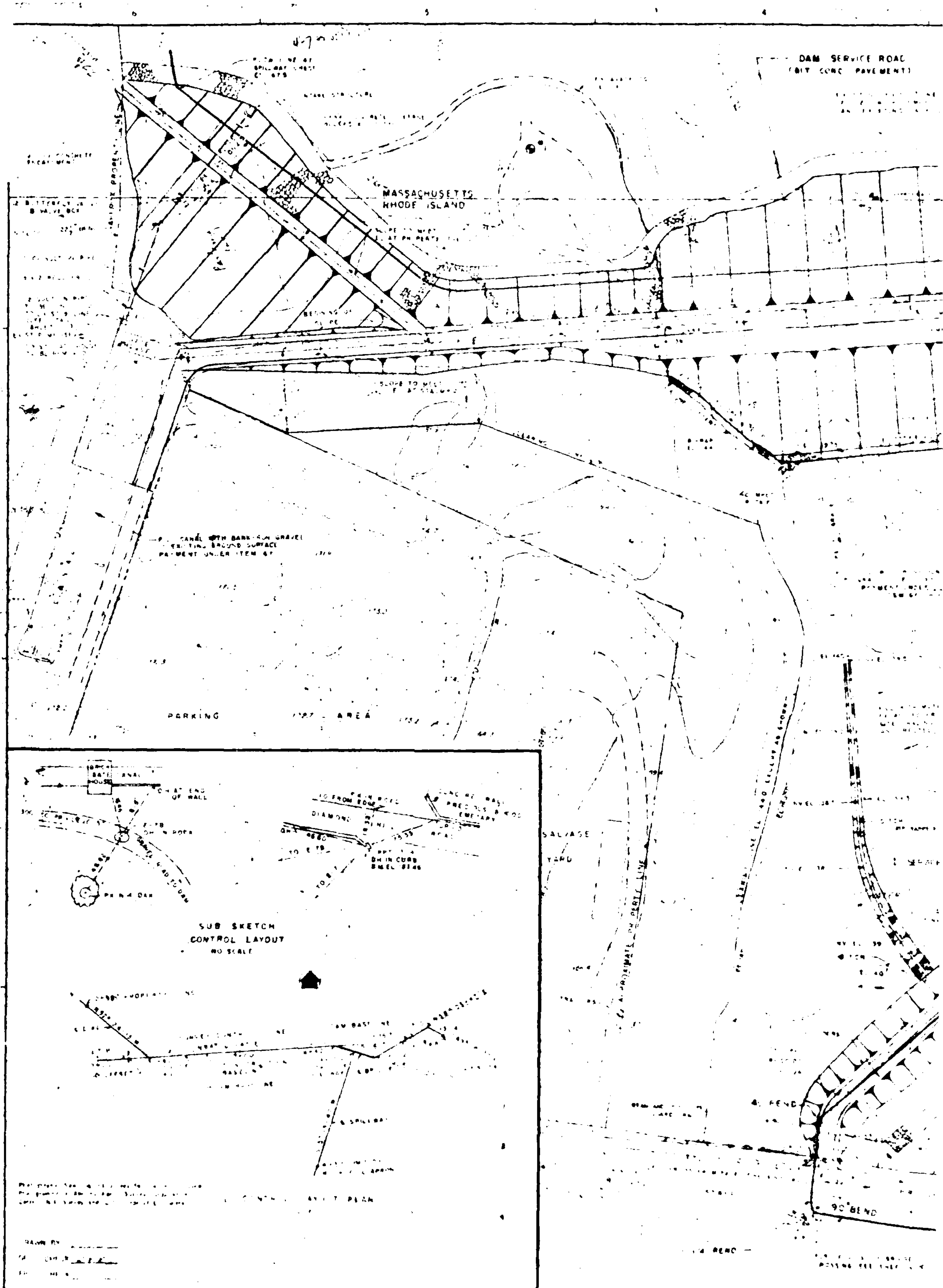
REGISTERED

MBE

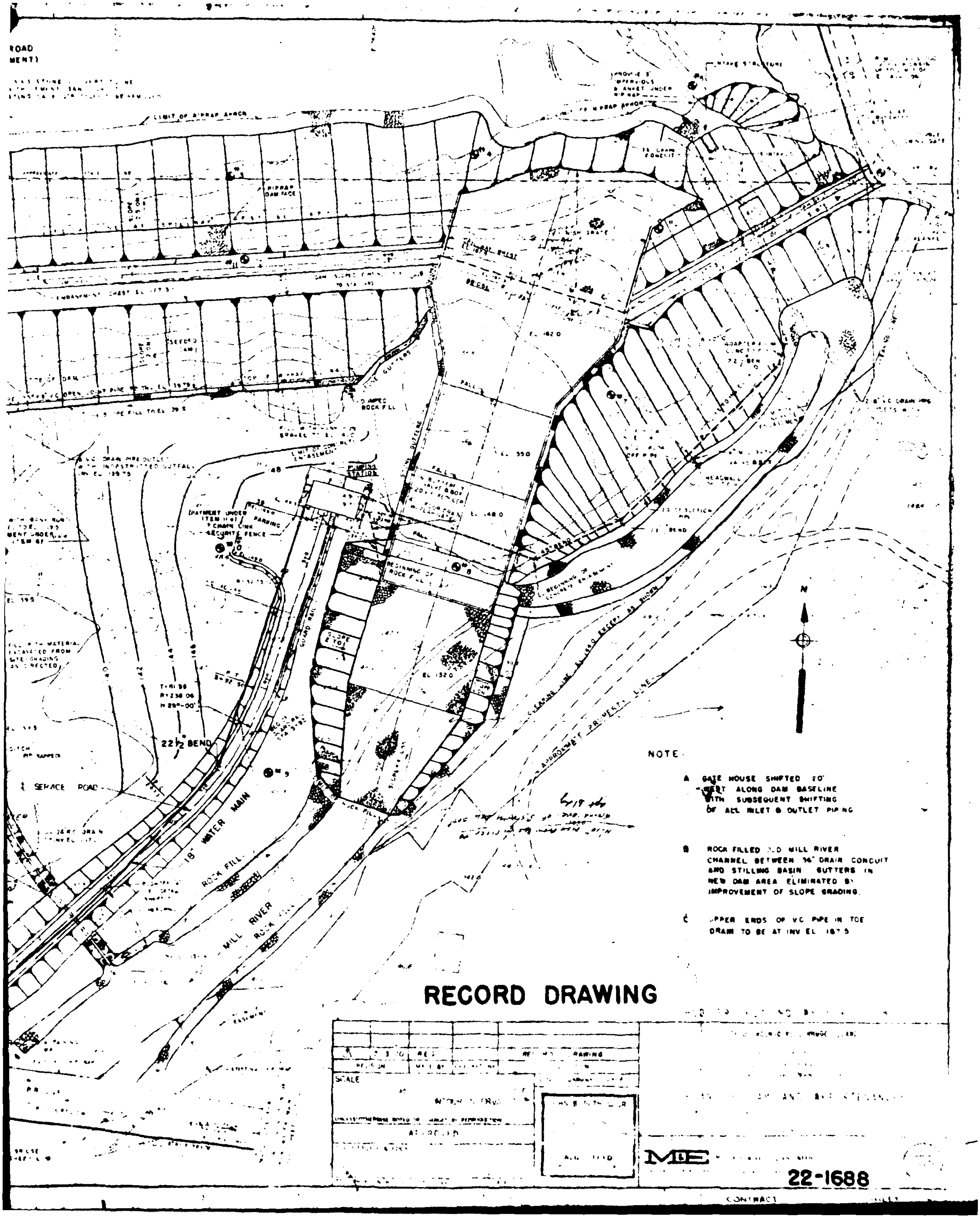
22-1688

CONTRACT

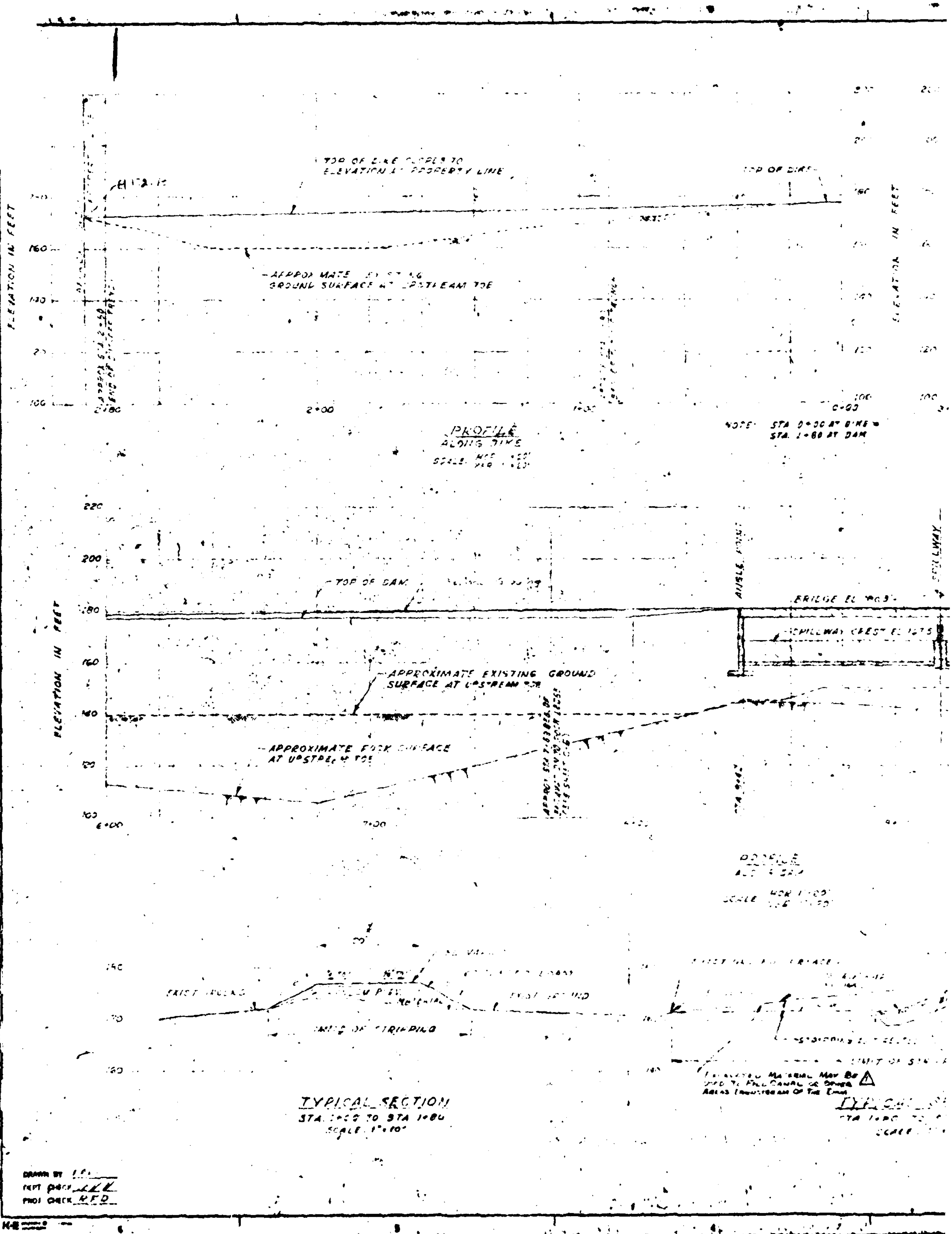
SHEET



DAM SERVICE ROAD
MASSACHUSETTS
RHODE ISLAND
PARKING
AREA
SALVAGE
YARD
SUB SKETCH
CONTROL LAYOUT
NO SCALE
90° BEND

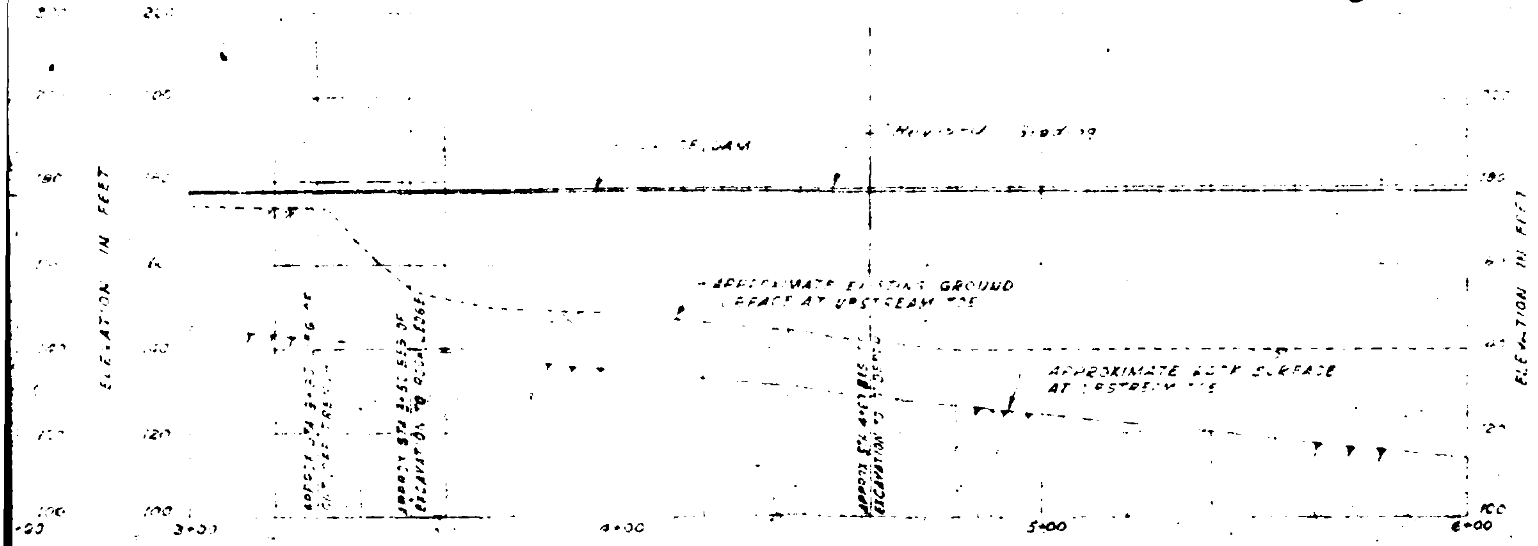


ROAD
LIMIT OF RIPRAP APRON
RIPRAP DAM FACE
EMBANKMENT CREST EL 177.5
SEEDED
INTAKE STRUCTURE
BRIDGE
EL 162.0
FALL
EL 150.0
EL 148.0
PUMPING STATION
PARKING
CHAIN LINK SECURITY FENCE
BEGINNING OF ROCK FILL
BEGINNING OF CONCRETE ENCASEMENT
EL 152.0
SLOPE 2 TO 1
GUARD RAIL
22½° BEND
SERVICE ROAD
18" WATER MAIN
ROCK FILL
MILL RIVER
APPROXIMATE PROPERTY LINE
N
NOTE
A GATE HOUSE SHIFTED 20' WEST ALONG DAM BASELINE WITH SUBSEQUENT SHIFTING OF ALL INLET & OUTLET PIPING
B ROCK FILLED OLD MILL RIVER CHANNEL BETWEEN 36" DRAIN CONDUIT AND STILLING BASIN. GUTTERS IN NEW DAM AREA ELIMINATED BY IMPROVEMENT OF SLOPE GRADING.
C UPPER ENDS OF VC PIPE IN TOE DRAIN TO BE AT INV EL 167.5
RECORD DRAWING
SCALE
APPROVED
22-1688
CONTRACT

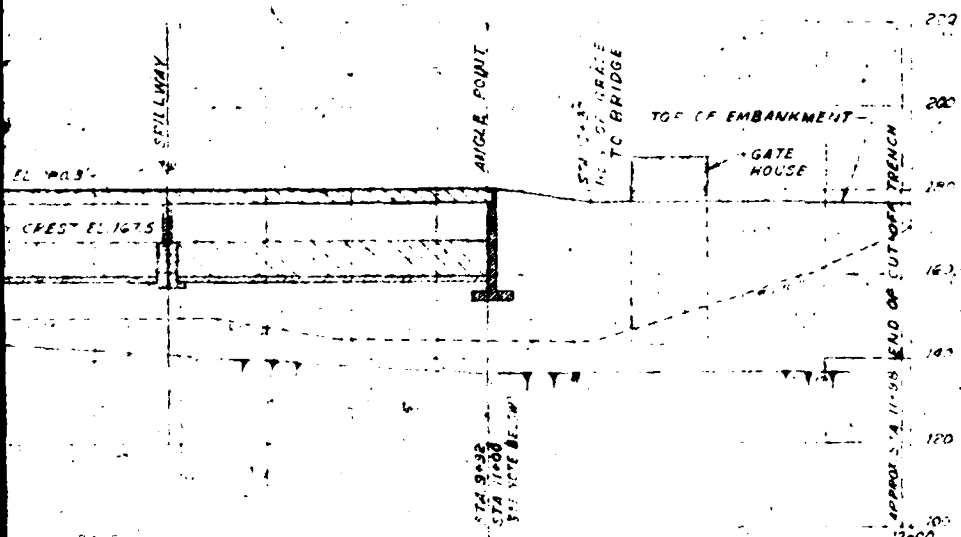


TOP OF DIKE
ELEVATION IN FEET
APPROXIMATE EXISTING GROUND SURFACE AT UPSTREAM TOE
PROFILE
ALONG DIKE
NOTE: STA 0+00 AT DIKE = STA. 2+60 AT DAM
TOP OF DAM
SPILLWAY
APPROXIMATE ROCK SURFACE AT UPSTREAM TOE
EXIST GROUND
TYPICAL SECTION
DRAWN BY
DEPT CHECK
PROJ CHECK

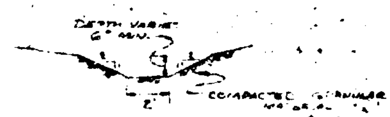
2



PROFILE
ALONG DAM
SCALE: HOR. 1" = 20' VER. 1" = 20'

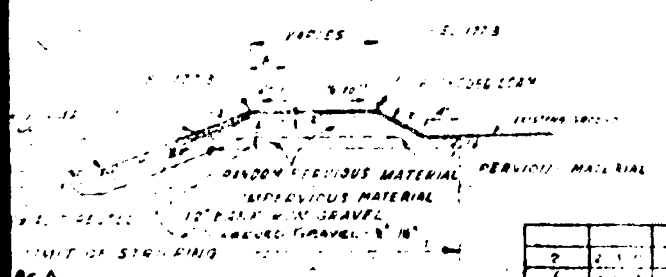


NOTE: STA 9+92 AT TOE OF SLOPE & STA 11+00 TOE OF SLOPE



TYPICAL DITCH SECTION

NOTE: APPROXIMATE ROCK SURFACE INTERPOLATED AS STRAIGHT LINES BETWEEN BORINGS.



TYPICAL SECTION

RECORD DRAWING

HUD PROJECT NO. WS-1-40-0008

2		HBH		RECORD DRAWING
REVISION		MADE BY	CHECKED BY	DESCRIPTION

SCALE AS SHOWN

DATE JANUARY, 1968

APPROVED

FOR METCALF & EDDY

JOHN S. BETHEL, JR. REGISTERED

CITY OF WOONSOCKET, RHODE ISLAND

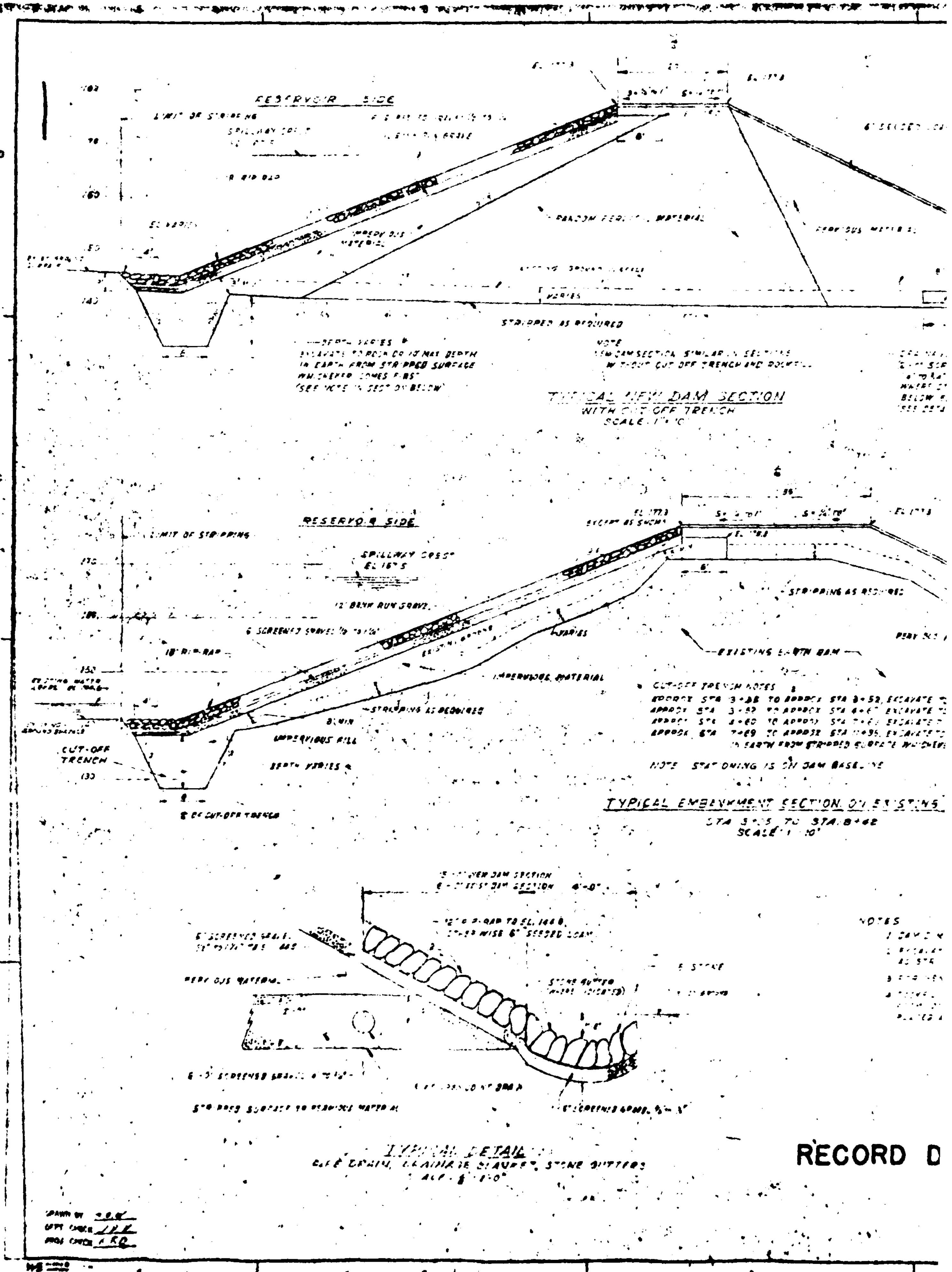
HARRIS POND DAM REHABILITATION, PUMPING STATION, RAW WATER MAIN AND APPURTENANT WORK

DAM EMBANKMENT SECTIONS AND PROFILE

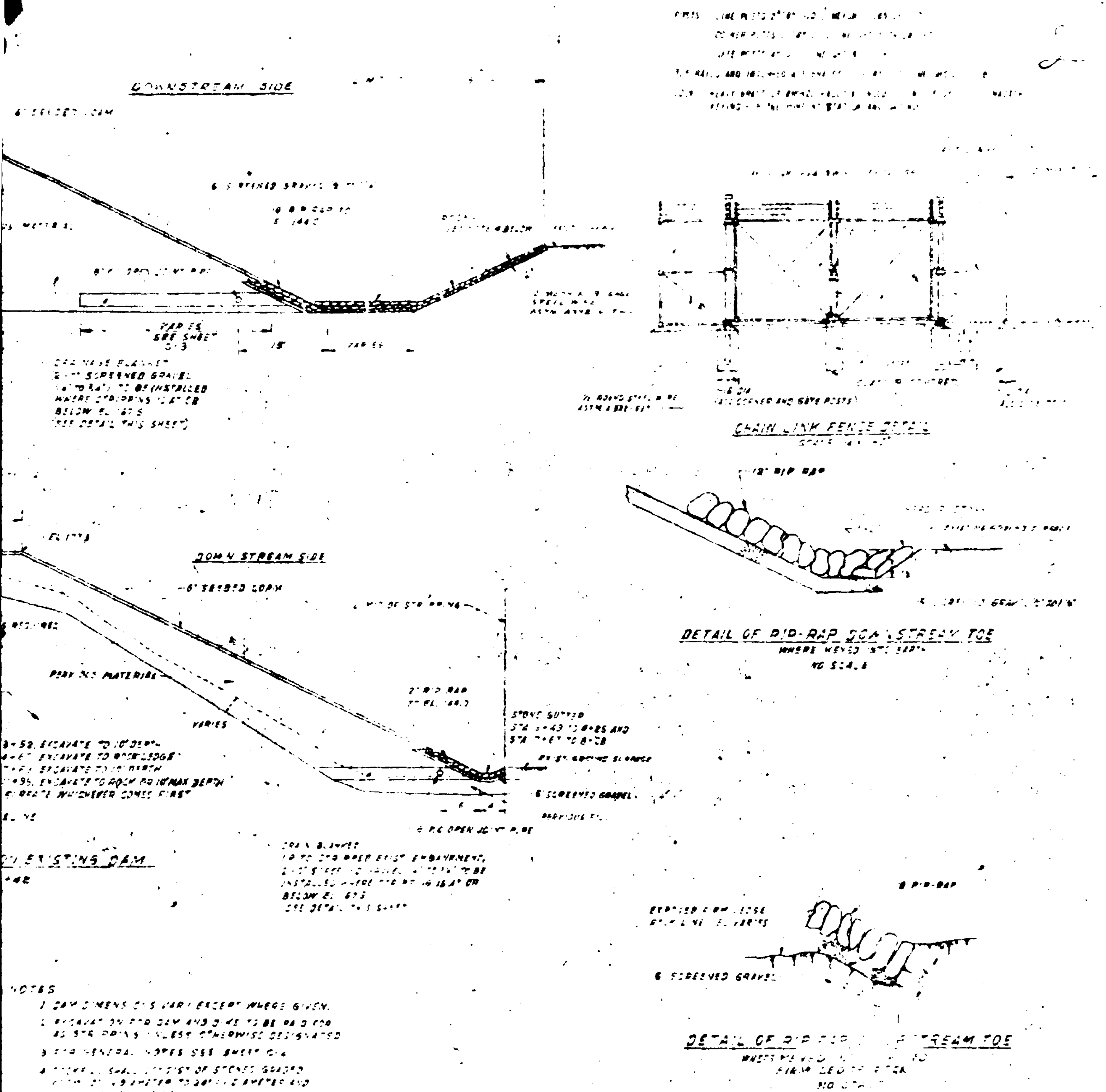
CIVIL

22-1688

CONTRACT 1967 P SHEET C-4



RESERVOIR SIDE
TYPICAL NEW DAM SECTION
WITH CUT OFF TRENCH
STRIPPED AS REQUIRED
RESERVOIR SIDE
LIMIT OF STRIPPING
SPILLWAY CREST
12" BANK RUN GRAVEL
18" RIP-RAP
STRIPPING AS REQUIRED
EXISTING EARTH DAM
IMPERVIOUS MATERIAL
IMPERVIOUS FILL
CUT-OFF TRENCH
DEPTH VARIES
CUT-OFF TRENCH NOTES
NOTE STATIONING IS ON DAM BASELINE
TYPICAL EMBANKMENT SECTION ON EXISTING
STONE GUTTER
PERVIOUS MATERIAL
TYPICAL DETAIL
NOTES
RECORD D



NOTES

1. DAM DIMENSIONS VARY EXCEPT WHERE GIVEN.
2. EXCAVATION FOR DAM AND DIKE TO BE PAID FOR AS STRIPPING UNLESS OTHERWISE DESIGNATED.
3. FOR GENERAL NOTES SEE SHEET C-4.
4. RIPRAP SHALL CONSIST OF STONES GRADED FROM ... DIAMETER TO ... DIAMETER AND PLACED AS SPECIFIED.

HUD PROJECT NO. WS-1-RI-0008

ORD DRAWING

1	7 5 70	REB		RECORD DRAWING
REVISION		MADE BY	CHECKED BY	DESCRIPTION

SCALE AS SHOWN

DATE JANUARY, 1968

UNLESS OTHERWISE NOTED OR CHANGED BY REPRODUCTION

APPROVED

FOR METCALF & EDDY

4-3-68

DATE

JOHN S. BETHEL, JR.

REGISTERED

CITY OF WOONSOCKET, RHODE ISLAND

... POND DAM REHABILITATION, PUMPING STATION, RAW WATER MAIN AND APPURTENANT WORK

DAM EMBANKMENT SECTIONS AND DETAILS

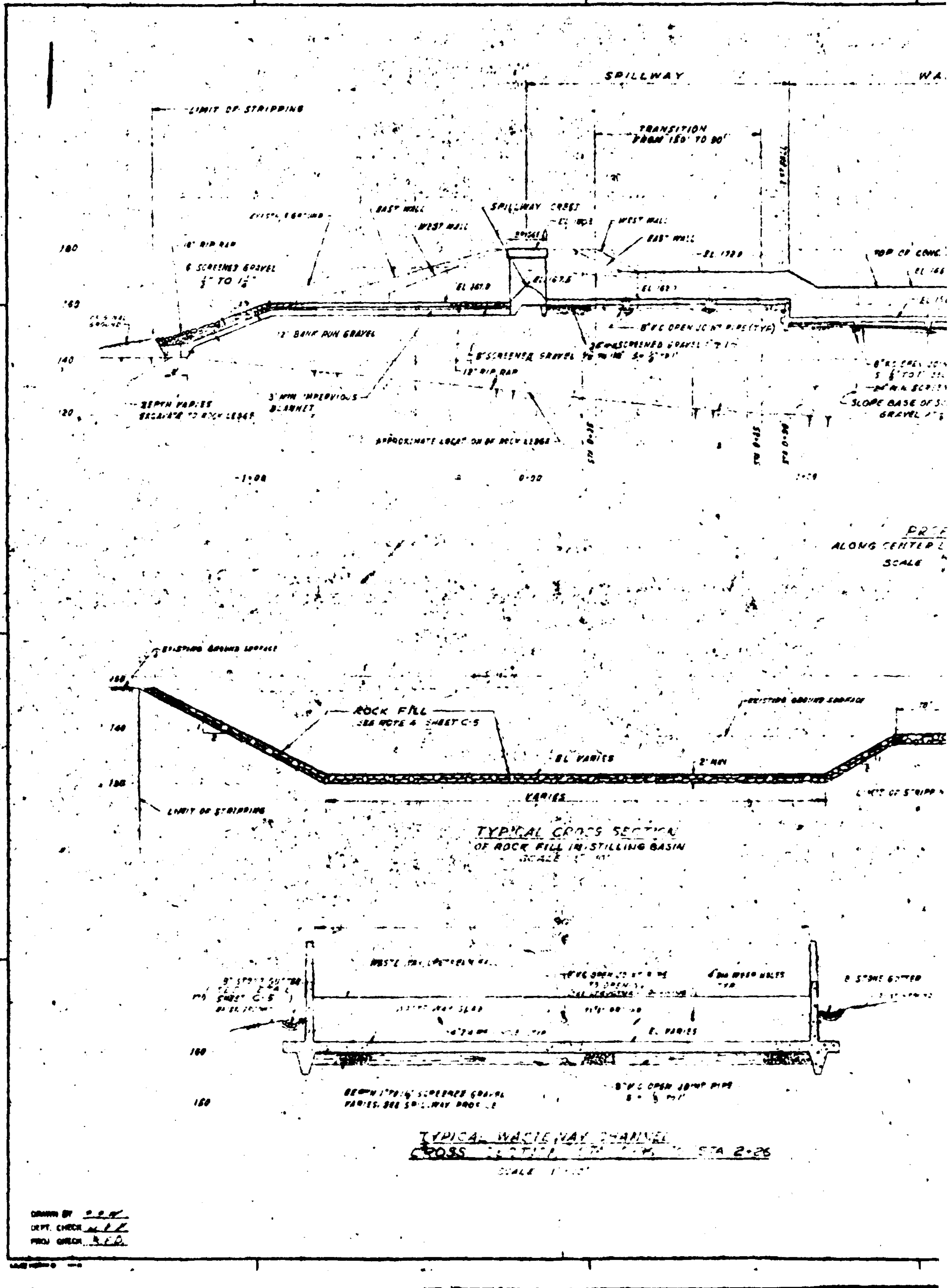
CIVIL

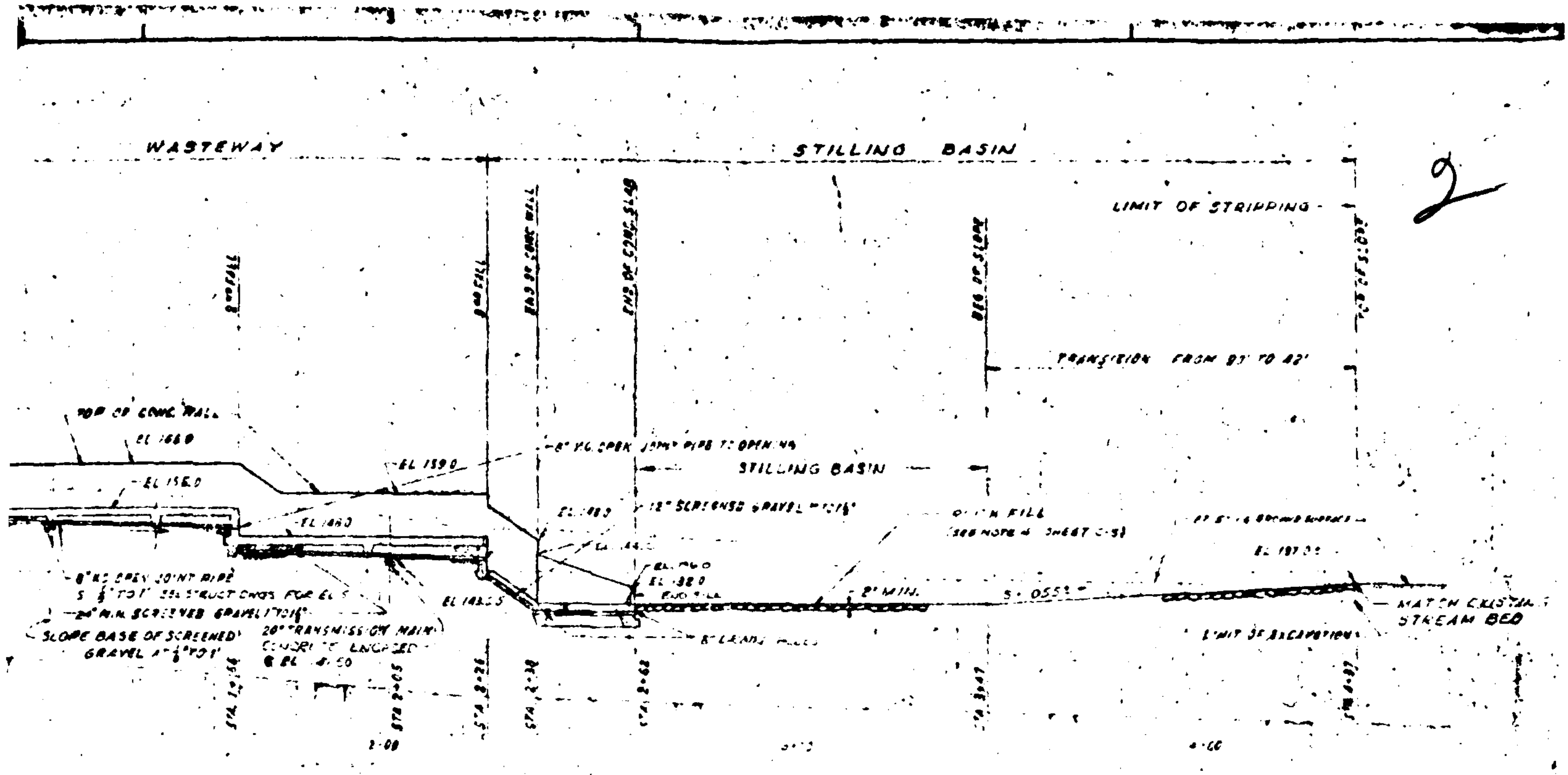
METCALF & EDDY | ENGINEERS

22-1688

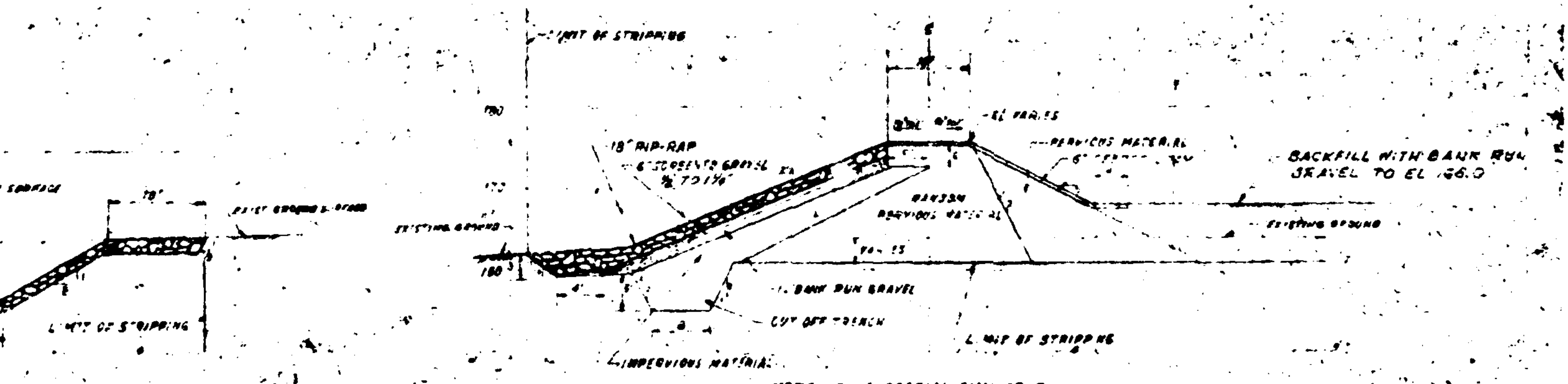
CONTRACT 1067-1

SHEET C-


SPILLWAY
LIMIT OF STRIPPING
TRANSITION FROM 150' TO 90'
SPILLWAY CREST
EAST WALL
WEST WALL
18" RIP RAP
6" SCREENED GRAVEL
12" BANK RUN GRAVEL
3' MIN. IMPERVIOUS BLANKET
DEPTH VARIES EXCAVATE TO ROCK LEDGE
APPROXIMATE LOCATION OF ROCK LEDGE
SLOPE BASE OF SCREENED GRAVEL
ALONG CENTER L
SCALE
ROCK FILL
SEE NOTE 4 SHEET C-5
EL VARIES
2' MIN
VARIES
LIMIT OF STRIPPING
EXISTING GROUND SURFACE
TYPICAL CROSS SECTION
OF ROCK FILL IN STILLING BASIN
WASTEWAY SLAB
EL VARIES
TYPICAL WASTEWAY CHANNEL
CROSS SECTION STA 2+26
DRAWN BY
DEPT. CHECK
PROJ. CHECK

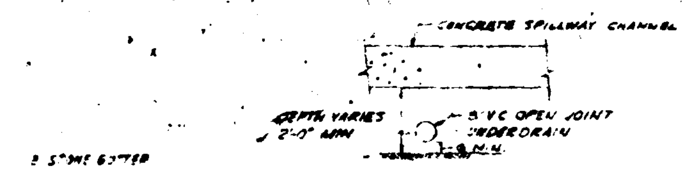


PROFILE
ALONG CENTER LINE OF SPILLWAY
SCALE HOR. 1"=20'
VER. 1"=20'



NOTE: DIKE SECTION SIMILAR IN SECTIONS WITHOUT CUT OFF TRENCH

TYPICAL DIKE SECTION
WITH CUT-OFF TRENCH
SCALE 1"=10'



TYPICAL UNDERDRAIN SECTION
NOT TO SCALE

RECORD DRAWING

HUD PROJECT NO. WS-1-RO-0008

A	2 3 70	REG		RECORD DRAWING
REVISION		MADE BY	CHECKED BY	DESCRIPTION

SCALE AS SHOWN

DATE JANUARY, 1968

JOHN S. BETHEL, JR.

REGISTERED PROFESSIONAL ENGINEER

APPROVED

FOR METCALF & EDDY

CITY OF WOONSOCKET, RHODE ISLAND

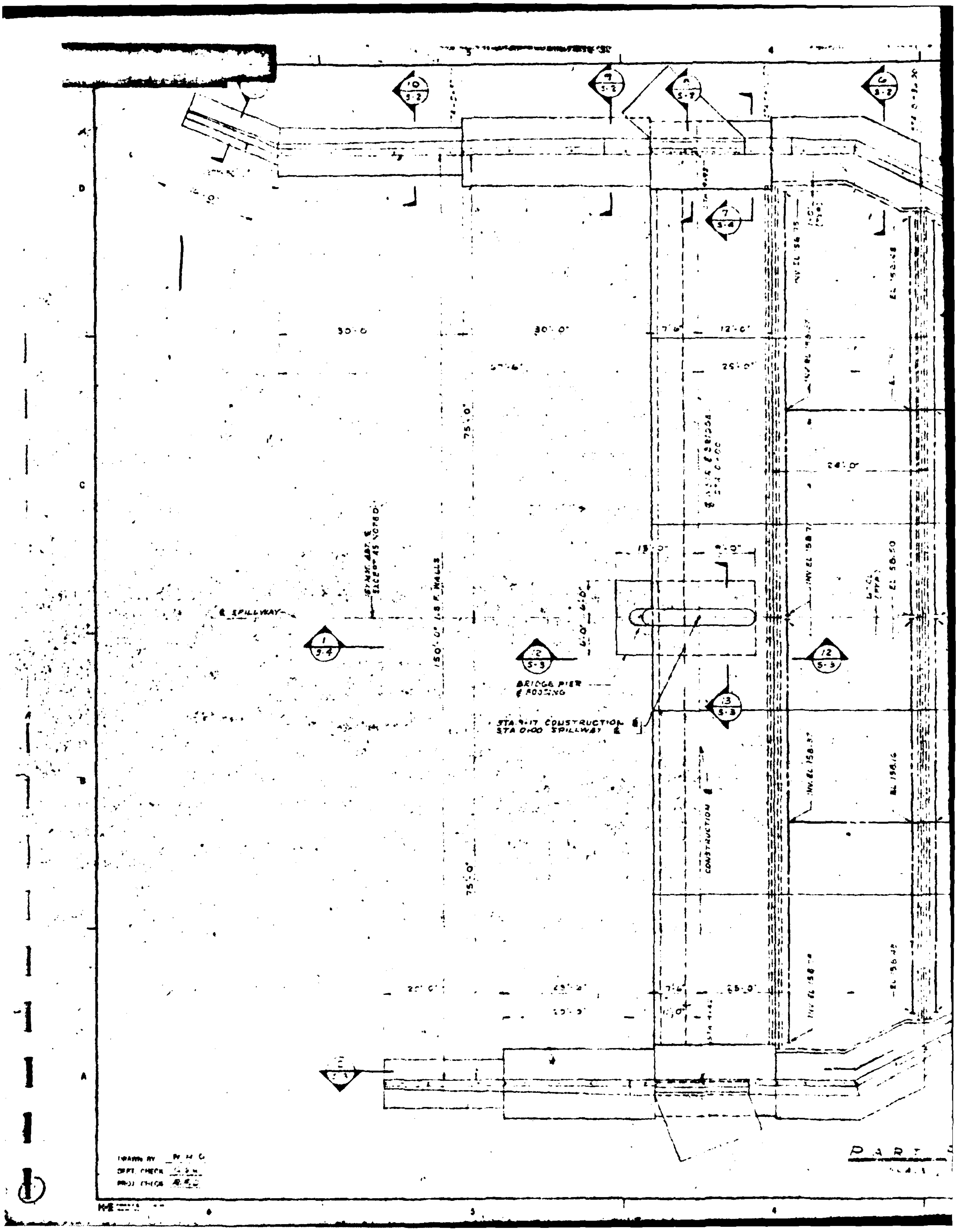
HARRIS POND DAM REHABILITATION, PUMPING STATION, RAW WATER MAIN AND APPURTENANT WORK

SPILLWAY PROFILE AND SECTIONS
DIKE SECTION

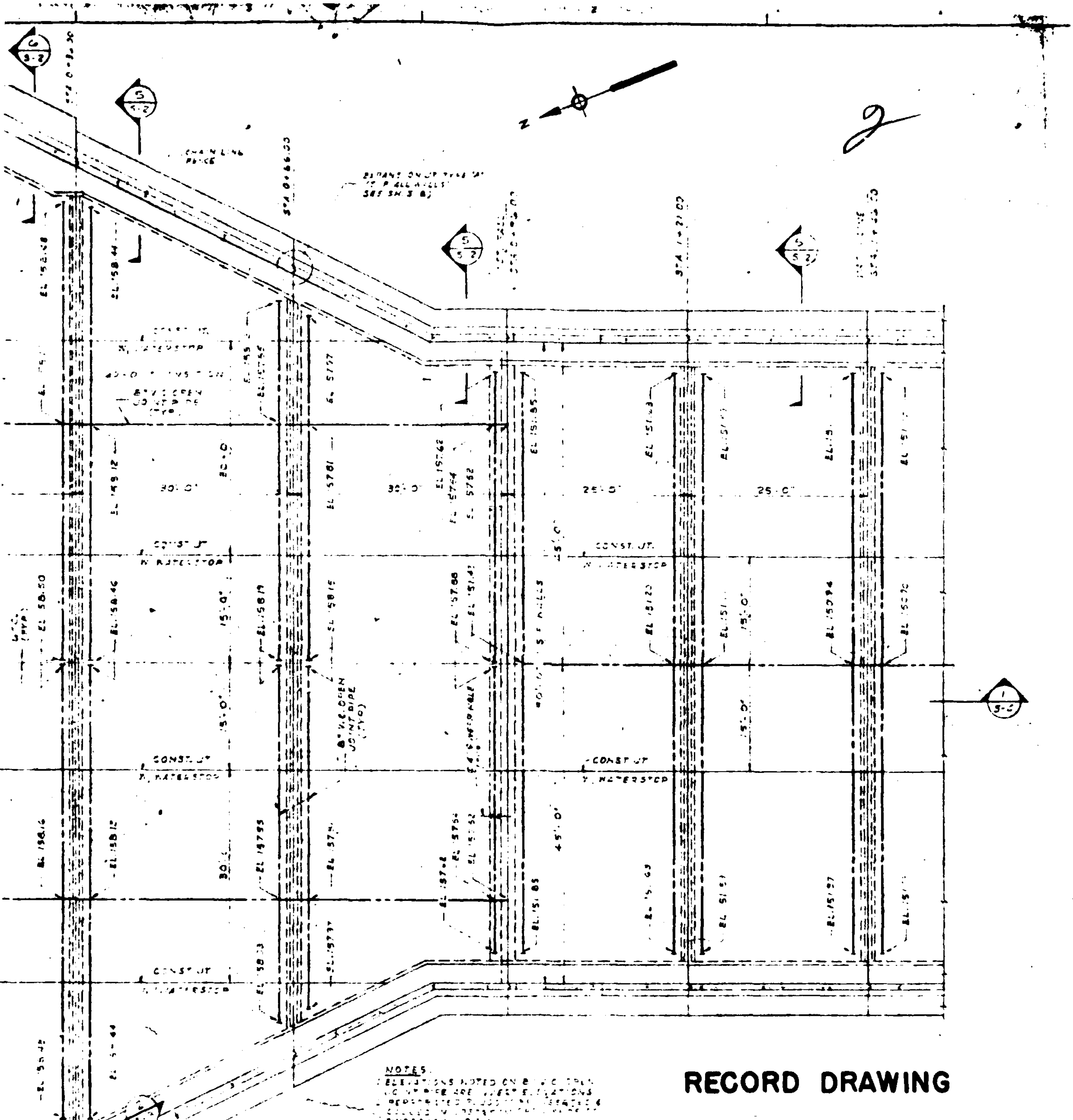
METCALF & EDDY ENGINEERS

22-1688

CONTRACT



30'-0"
12'-0"
7'-6"
25'-0"
24'-0"
75'-0"
150'-0" I.B.F. WALLS
SYMM. ABT ℄ EXCEPT AS NOTED
℄ SPILLWAY
BRIDGE PIER ℄ FOOTING
STA 9+17 CONSTRUCTION ℄
STA 0+00 SPILLWAY ℄
CONSTRUCTION ℄
DRAWN BY
DEPT. CHECK
PROJ. CHECK



NOTES:

RECORD DRAWING

HUD PROJECT NO. WS-...-NC-0008

CITY OF WOONSOCKET, RHODE ISLAND

HARRIS POND DAM REHABILITATION
PUMPING STATION, RAW WATER MAIN
AND APPURTENANT WORK

SPILLWAY PLAN I

REVISION		MADE BY	

SCALE

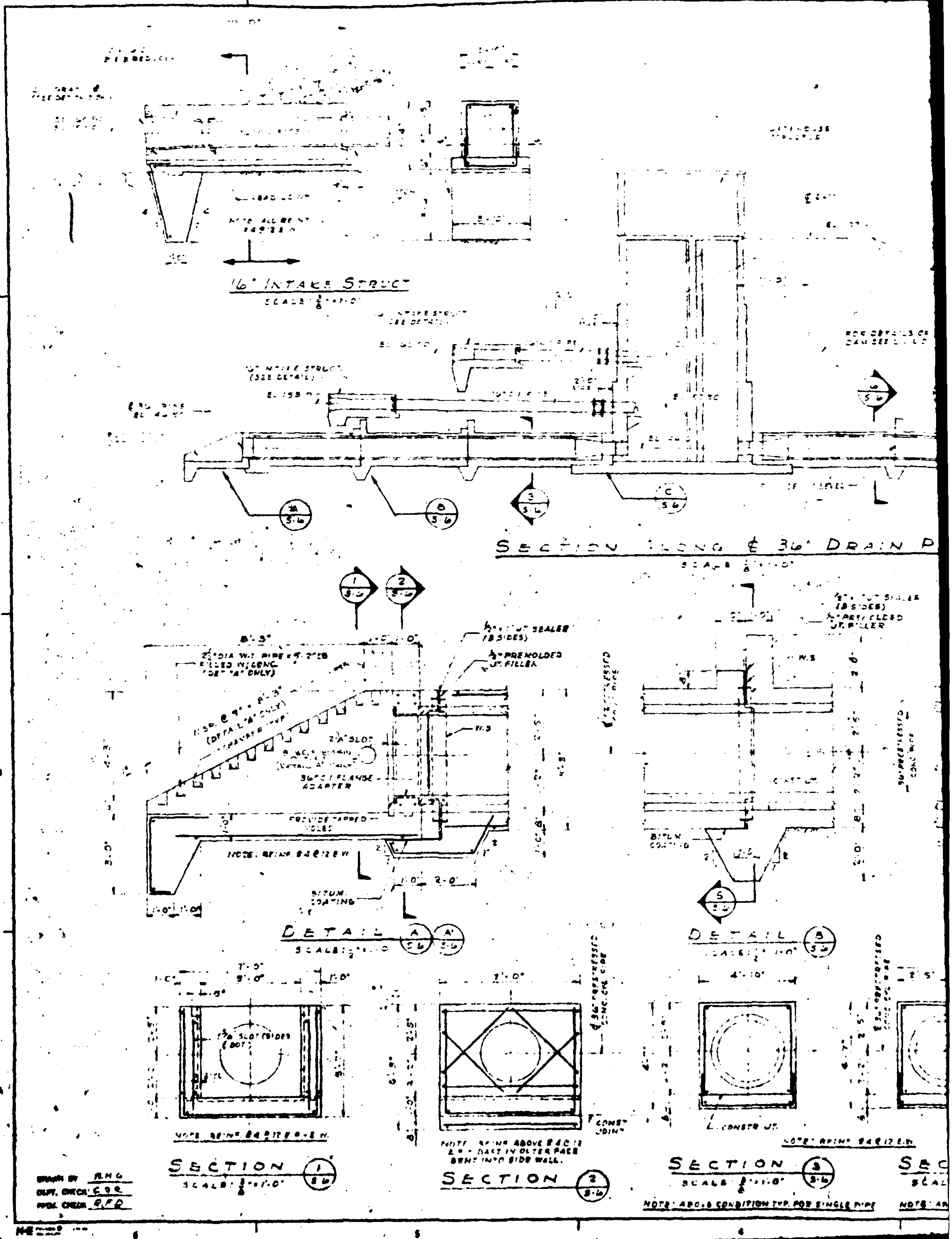
DATE JANUARY 1968

REGISTERED

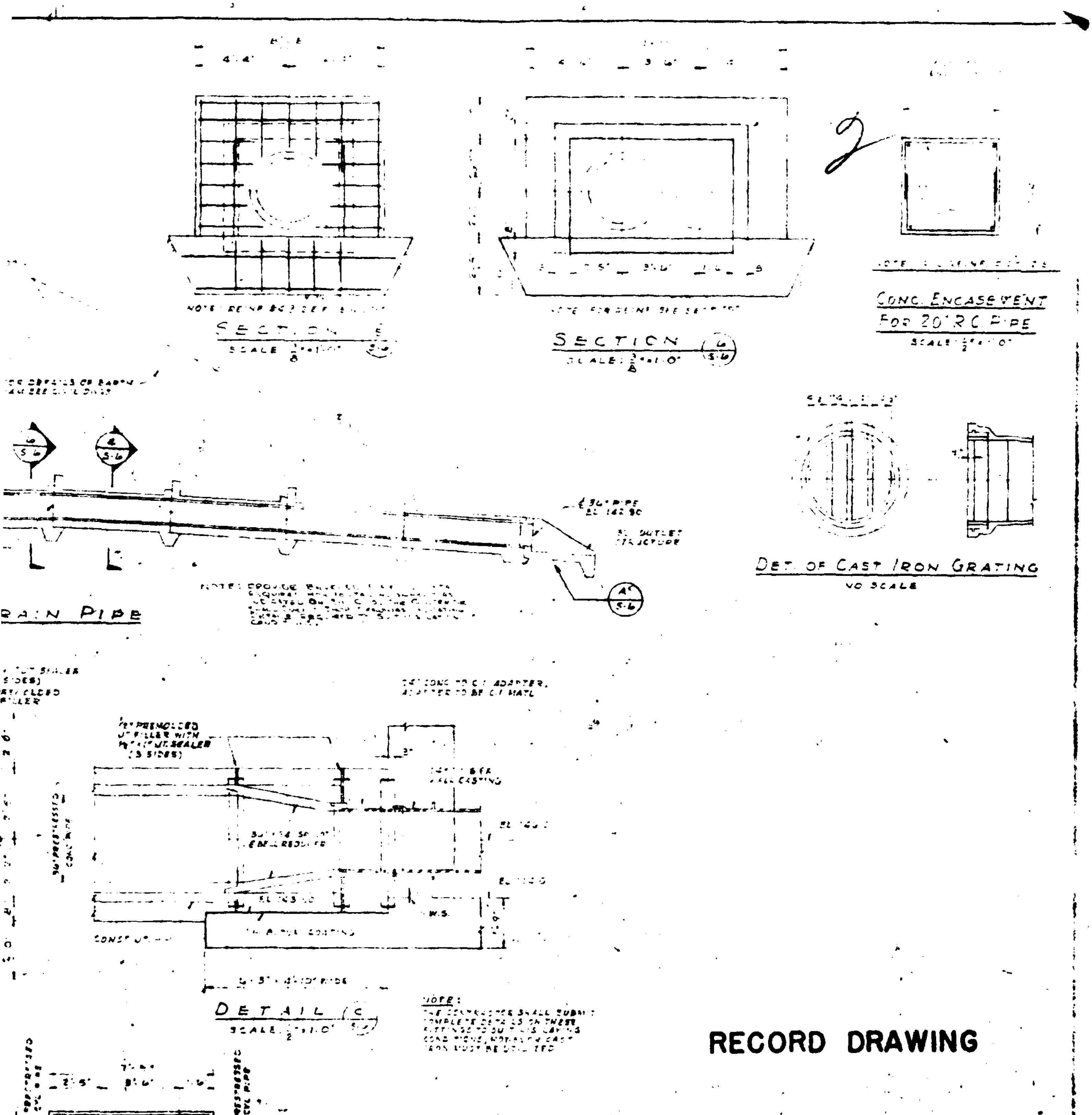
METCALF & EDDY ENGINEERS

22-1688

CONTRACT SHEET



16" INTAKE STRUCT
SCALE 3/8" = 1'-0"
SECTION ALONG ℄ 36" DRAIN P
SCALE 1/8" = 1'-0"
DETAIL A A'
SCALE 1/2" = 1'-0"
NOTE: REINF. #4 @ 12 E.W.
PROVIDE TAPPED HOLES
36" C.I. FLANGE ADAPTER
BITUM. COATING
1/2" PREMOLDED JT. FILLER
(3 SIDES)
DETAIL B
SCALE 1/2" = 1'-0"
SECTION 1
SCALE 3/8" = 1'-0"
NOTE: REINF. #4 @ 12 E.W.
SECTION 2
NOTE: REINF. ABOVE #4 @ 12 E.W. BARS IN OUTER FACE BENT INTO SIDE WALL.
CONST JOINT
SECTION 3
SCALE 3/8" = 1'-0"
CONSTR. JT.
NOTE: REINF. #4 @ 12 E.W.
NOTE: ABOVE CONDITION TYP. FOR SINGLE PIPE
DRAWN BY R.H.G.
DEPT. CHECK C.D.C.
PROJ. CHECK R.F.Q.



RECORD DRAWING

HUD PROJECT NO. WS-1-40-0008

CITY OF WOONSOCKET, RHODE ISLAND

HARRIS POND DAM REHABILITATION
PUMPING STATION, RAW WATER MAIN
AND APPURTENANT WORK

INTAKE AND OUTLET
SECTIONS AND DETAILS

STRUCTURAL

		MADE BY	CHECKED BY	DESCRIPTION

SCALE AS SHOWN

DATE JANUARY, 1968

APPROVED

FOR METCALF & EDDY

4-3-68

REGISTERED

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22-1688

CONTRACT 1967-1

SHEET S-6

State of Rhode Island and Providence Plantations

DAMS AND RESERVOIRS

Application for Approval
of Plans

April 11 1968

All plans submitted to the DIVISION OF HARBORS AND RIVERS are required by Section 5-S-26, of the General Laws of 1956, to be stamped with the seal of a "Registered Professional Engineer"

Chief of the Division of Harbors and Rivers
~~214 State Office Building~~
~~Providence, R. I.~~
106 Veterans Memorial Building
Providence, R. I 02903

Dear Sir:

The undersigned respectfully requests the approval of the plans and specifications, herewith submitted, for the ~~construction~~ ~~alteration~~ of a dam to be built in the ~~town~~ city of Woonsocket, Providence County, Rhode Island on Mill river.

(The applicant is to fill in the following items)

1. The drainage area at this dam is 34.7 square miles.
2. The spillway capacity of this dam at maximum discharge, is 8500* cubic feet per second.
3. Waste gate discharge, under freshet conditions is 200 cubic feet per second.
4. The estimated greatest freshet flow at this dam is 3300** cubic feet per second.

*Based on flood routing of Corps of Engineers' Standard Project Flood for the Mill River.
**Estimated flood of record for the Mill River(without 1955 Harris Pond Dam failure).

Plans (in triplicate) ~~xxxxxx~~ showing details of construction and locality, and specifications of construction of the proposed work must accompany this petition. Also plan of property involved and proof of ownership.

Owner's signature A. Edgar Lussier, Mayor

Address 169 Main Street
Woonsocket, Rhode Island *Tel.* 754-5400

(See other side)

August 23, 55

Henry Isé, Chief, Division of Harbors & Rivers

Public Works

John V. Keily, " " " " "

Public Works

Failure of Dams on Mill River at Woonsocket, R. I.

Yesterday I visited Harris Pond Dam #73 at Woonsocket, R. I. This earth dam had breached and the pond was virtually empty. This disaster was caused by the failure of the Spindleville Dam near Hopedale, Mass., about seven miles upstream from Harris Pond on the Mill River. This flood of water, occuring at the time of a heavy flow caused by hurricane "Diane", caused the water in Harris Pond to top the trench embankment leading to the Horseshoe Falls so-called and to erode the earth embankments until the east end of the main dike on Harris Pond gave away and left a breach between 100 and 200 feet wide. This torrent followed the course of the Mill River and washed out bridges on Privilege and School Streets directly below. There was a low dam across the Mill River some distance north of School Street, but it was impossible to reach this site due to tangled trees, etc. This Dam #75 may have been washed away. Below School Street, another earth dam at Social Pond (Dam #72) has also breached for a 60 foot width and the pond emptied into the Social area of Woonsocket, before reaching the Blackstone River. There were heavy deposits of sand and gravel on many areas until the Blackstone River was reached. In fact the entire stream bed of the Mill River is filled with gravel above and below School Street, and the river is now flowing thru a a trench to the west of this river and thence into Social Pond.

We viewed the dams across the Blackstone River below Woonsocket, namely Manville #59, Albion #60, Ashton #61 and all seemed to be carrying the flood in a satisfactory manner. Minor floods had taken place at each location mainly due to overtopping of trench dikes. At Manville a weave-shed across the river and foundations under part of the wall had been washed away and have suffered serious damage.

At Pratt Dam #62 at Lonsdale,, the flood had taken out a section of the railroad, breached the trench to No. 4 Mill, and washed out part of the State Highway on Lonsdale Avenue. Considerable damage was done to small industries on the Blackstone River below this dam.

At Pawtucket the water is claimed to be 2½ feet higher than any previously recorded flood. Flooded basements were the chief casualties in Pawtucket.

Apprarently it was the breaking of the dam at Hopedale, Mass. that sent a flood wave down the Mill River on top of an already flloded condition. This wave over-taxed the existing earth structures at the various locations down stream and caused them to fail.

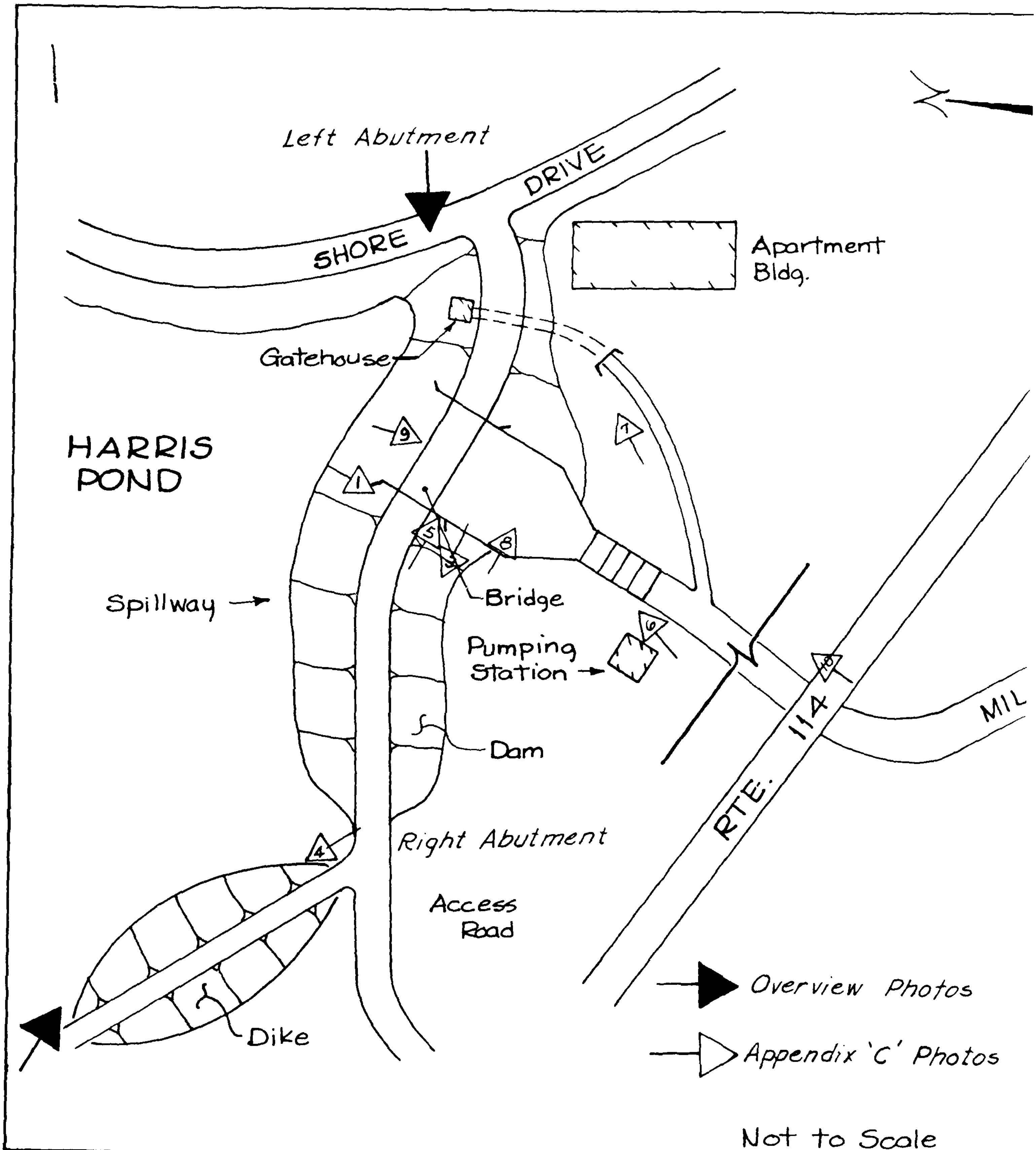
It can be readily seen that a large factor of safety will be required when these structures are rebuilt.

About 30 pictures were taken of the flood conditions some 72 hours after the peak of the flood.

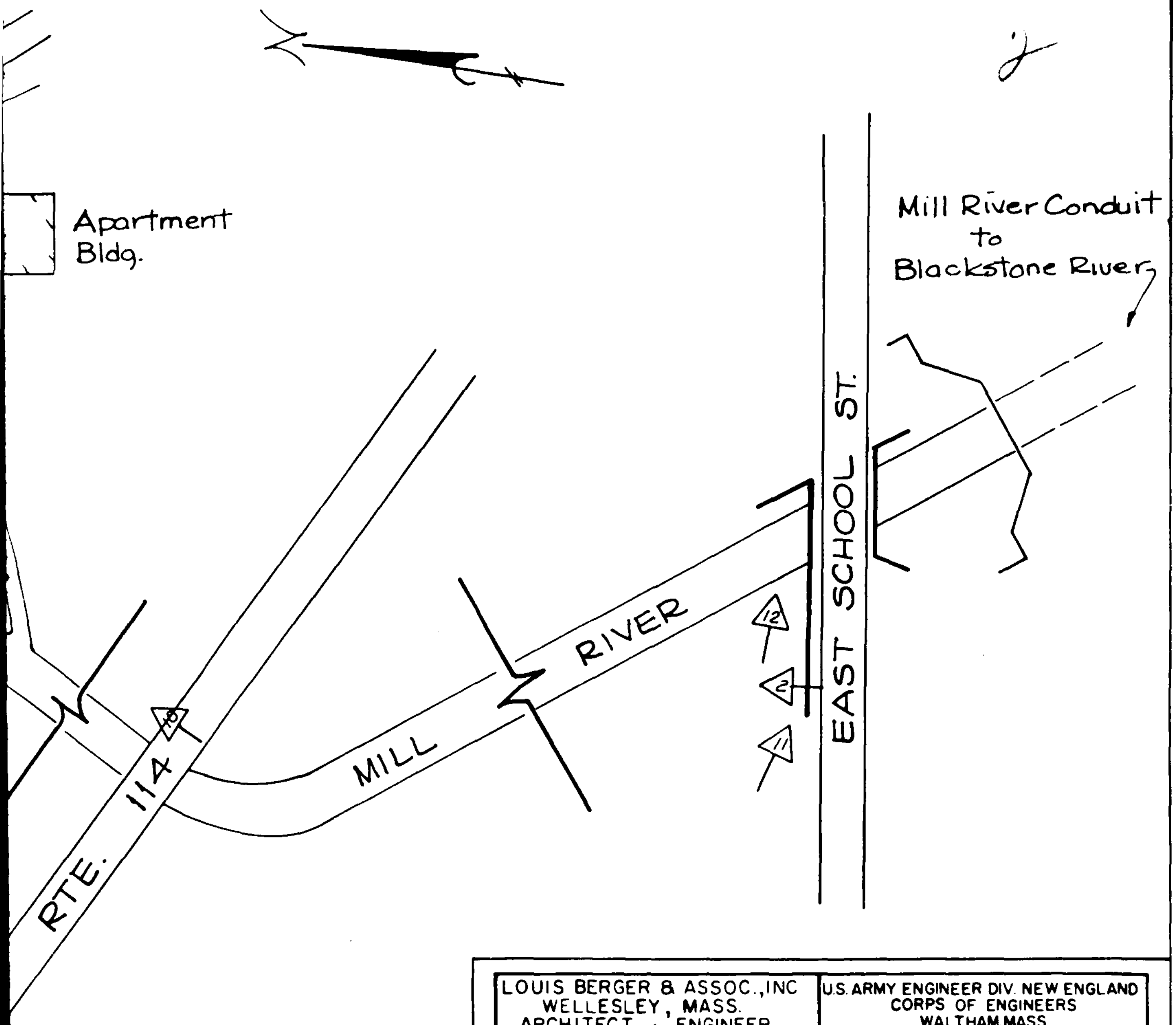
John V. Keily

APPENDIX C

SELECTED PHOTOGRAPHS



Left Abutment
SHORE DRIVE
Apartment Bldg.
Gatehouse
HARRIS POND
Spillway
Bridge
Pumping Station
Dam
RTE. 114
MIL
Right Abutment
Access Road
Dike
Overview Photos
Appendix 'C' Photos
Not to Scale



Overview Photos

Appendix 'C' Photos

Not to Scale

LOUIS BERGER & ASSOC., INC WELLESLEY, MASS. ARCHITECT · ENGINEER	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

HARRIS POND DAM

SKETCH PLAN SHOWING LOCATION & ORIENTATION OF PHOTOS

STATE - R.I.

			SCALE
			DATE

HARRIS POND DAM

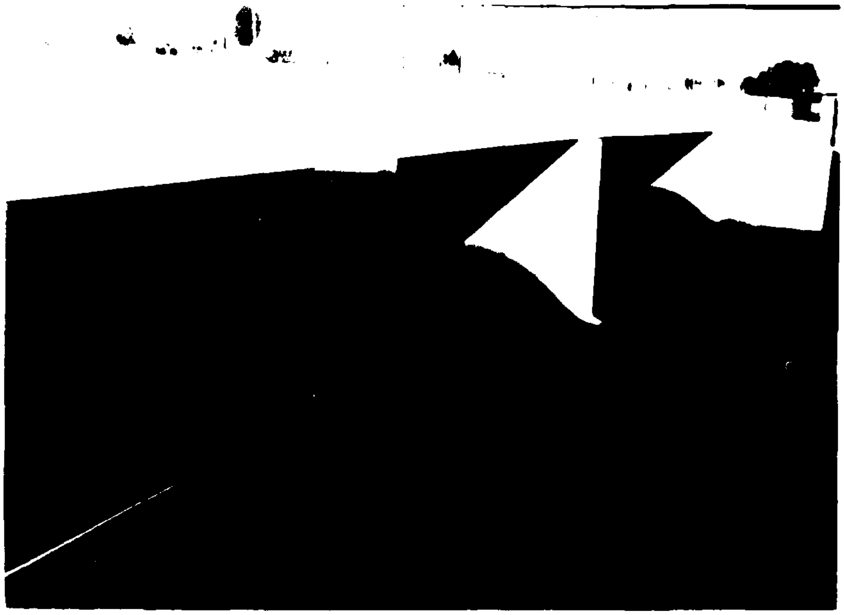
1. Upstream slope at left training wall to spillway.

2. Mill River channel looking upstream from E. School Street.

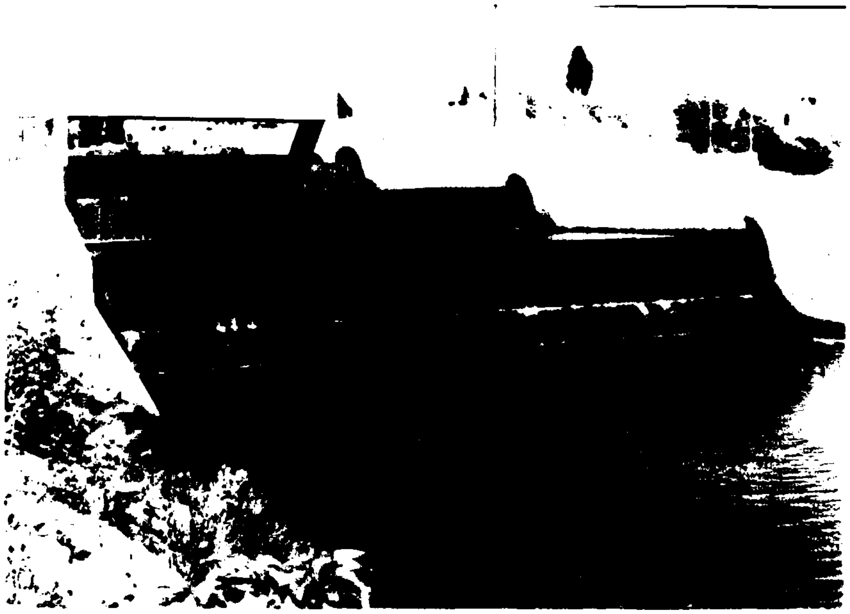
3. Downstream slope from right side of spillway.

4. Upstream slope of dike at right abutment.

HARRIS POND DAM

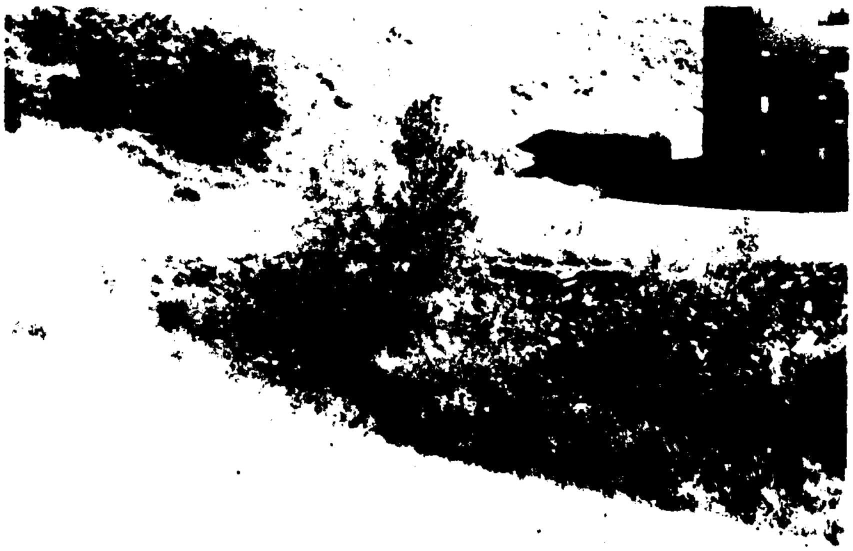


5. Spillway weir from right training wall.



6. Stilling basin from right side of downstream channel.

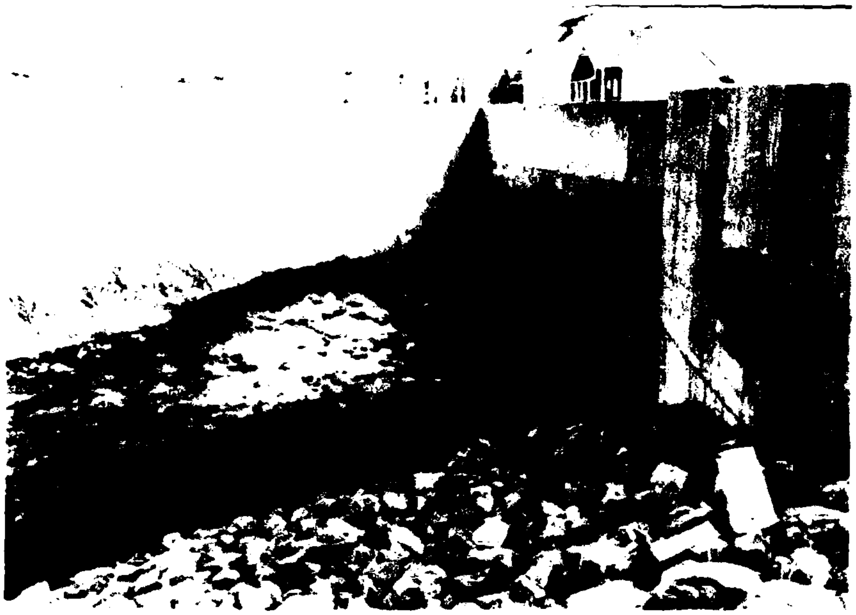
7. 36 in. dia. outlet pipe.



8. Downstream channel from 36 in. dia. outlet pipe.

9. Stilling area and downstream channel from spillway bridge.

10. Downstream channel from Route 114 bridge looking towards spillway.

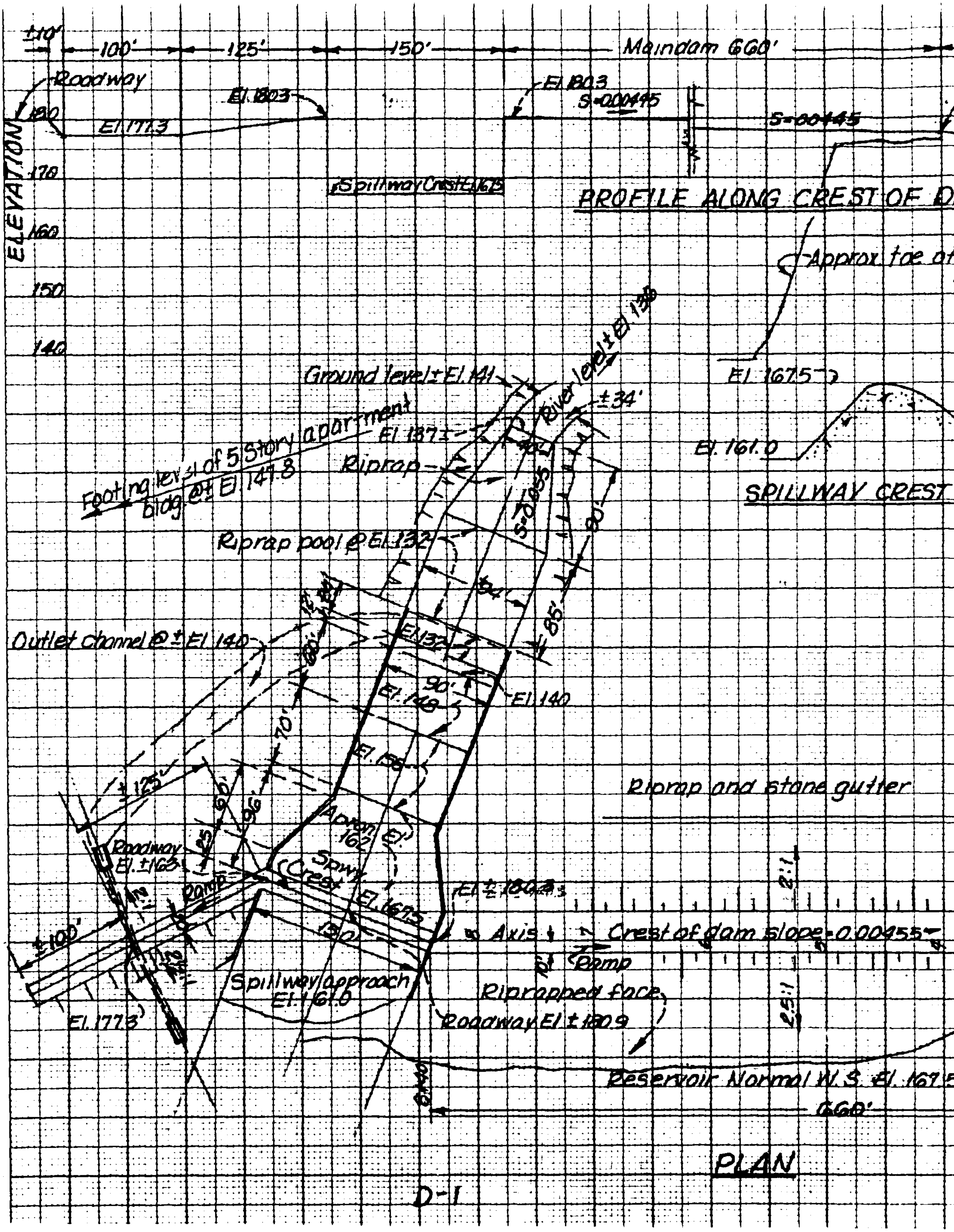


11. Entrance to Mill River conduit under East School Street.

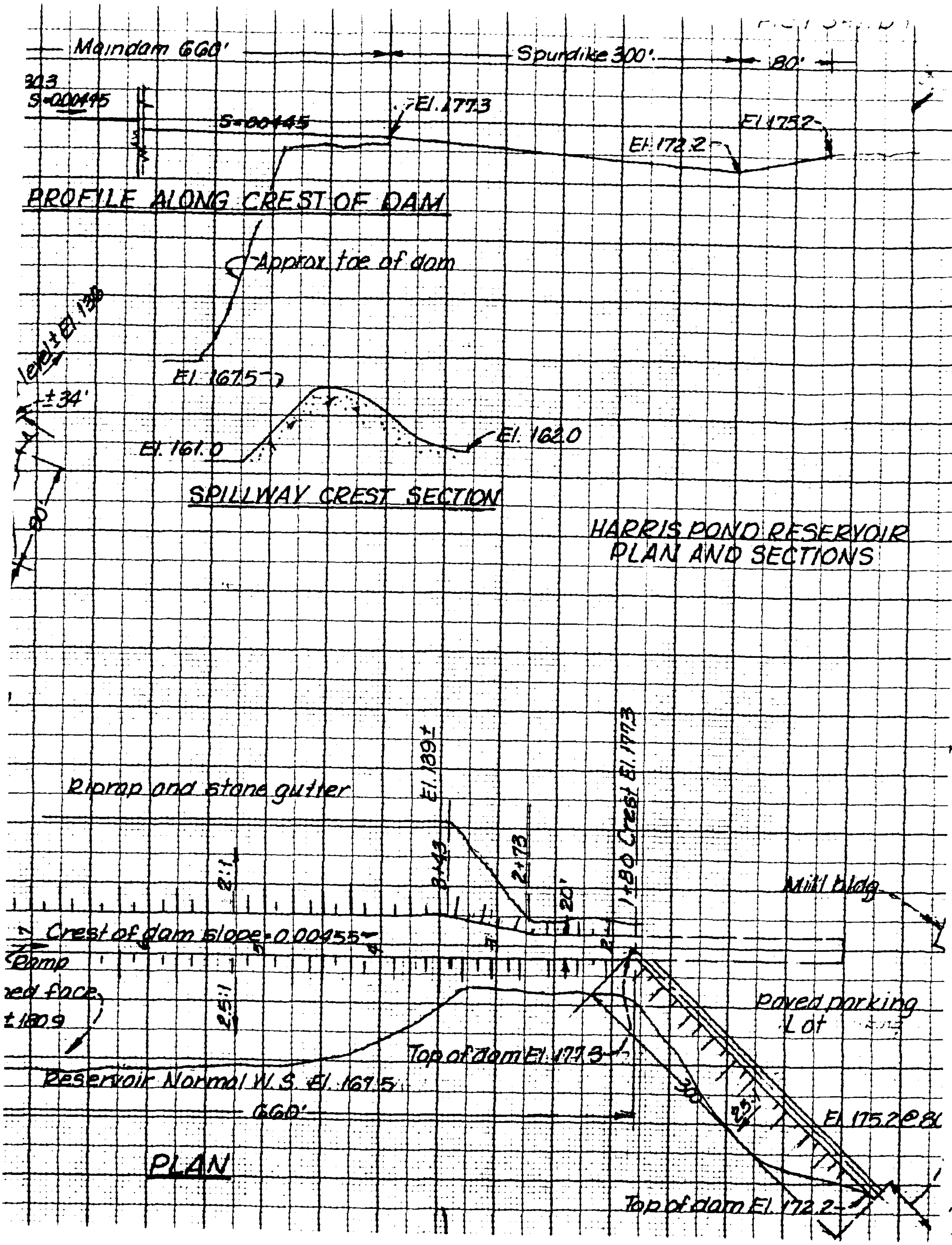
12. E. School St. and entrance to Mill River conduit (center) to Blackstone River.

APPENDIX D

HYDROLOGIC & HYDRAULIC COMPUTATIONS



±10'
100'
125'
150'
Maindam 660'
Roadway
El.180.3
El.177.3
El.180.3
S=0.00445
S=.00445
ELEVATION
180
170
160
150
140
Spillway Crest El.167.5
PROFILE ALONG CREST OF D
Approx toe of
El. 167.5
El. 161.0
SPILLWAY CREST
Ground level ± El. 141
River level ± El. 136
±34'
Footing level of 5 Story apartment bldg. @ ± El. 147.8
El. 137 ±
Riprap
Riprap pool @ El. 132
S=0.055
90'
54'
85'
Outlet channel @ ± El. 140
El. 132
90'
El. 140
El. 148
70'
El. 156
± 125'
Riprap and stone gutter
Apron El. 162
Roadway El. ±163
Ramp
Spwy Crest El. 167.5
El. ± 180.3
Axis
Crest of dam slope 0.00455
Ramp
±100'
130'
Spillway approach El. 161.0
Riprapped face
Roadway El ± 180.9
El. 177.3
2:1
2.5:1
Reservoir Normal W.S. El. 167.5
660'
PLAN


Maindam 660'
Spurdike 300'
80'
303
S=0.00445
S=.00445
El. 177.3
El. 175.2
El. 172.2
PROFILE ALONG CREST OF DAM
Approx. toe of dam
El. 167.5
±34'
El. 161.0
El. 162.0
90
SPILLWAY CREST SECTION
HARRIS POND RESERVOIR
PLAN AND SECTIONS
Riprap and stone gutter
El. 189±
1+80 Crest El. 177.3
3+43
2+73
2:1
20'
Mill bldg.
Crest of dam slope=0.00455
Ramp
2.5:1
Paved parking
Lot
Top of dam El. 177.3
Reservoir Normal W.S. El. 169.5
660'
30
El. 175.2 @ 80
PLAN
Top of dam El. 172.2

BY________ DATE [illegible]

CHKD. BY________ DATE________

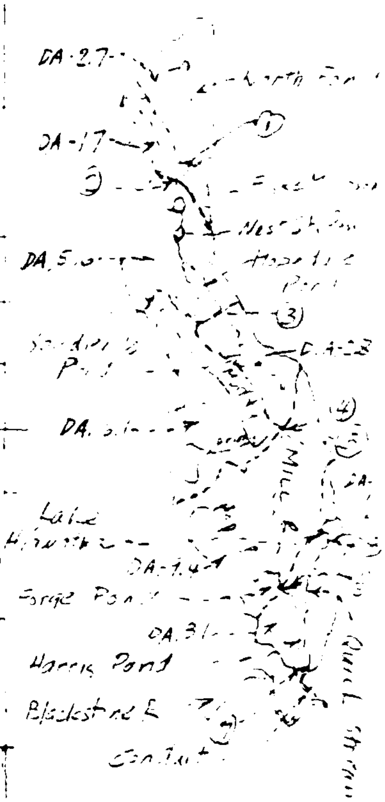
SUBJECT [illegible]

LOUIS BERGER & ASSOCIATES INC.

SHEET NO. D-2 OF____

PROJECT________

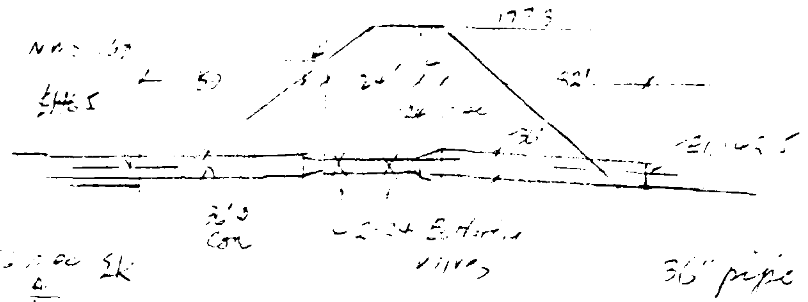
	River site	Sub Drainage Area sq. mi.	Reservoir Area Acres	Elev.	Average River Slope upstream %	Average River Slope upstream ft/mi	[illegible]	Between sites Length mi	Between sites Slope ft/mi
1	North Pond Dam	2.0	232	340	.0058	30.5	0.72	-	
2	Confluence Mill + unnamed stream	1.7	-	300	.0136	71.8	2.02	0.87	46.0
3	[illegible] Pond Dam	5.6	72	274	.0018	9.4	2.61	3.84	9.4
4	Confluence Mill R. + Muddy Brook	2.8	-	198.5	[illegible]	34.2	3.40	1.14	34.2
5	Muddy Brook Above Mill R. Confluence	6.1	-	198.5	[illegible]	22.2	5.22	1.82	22.2
6	Mill R. From Muddy Confluence to Forge Pond	9.4	10	177	.0007	32.0	4.44	4.23	[illegible]
7	From Forge Pond + Quick River to Harris Pond Dam	3.1	111	135	-			2.0	70.0
		31.4							
	[illegible]	1.1	50	229	.0135	71.0	1.96	-	-
	[illegible]			158	0.0153	33.0	2.59	1.25	53.3
		31.5							



BY [illegible] DATE 2-3-79 **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. D-3 OF

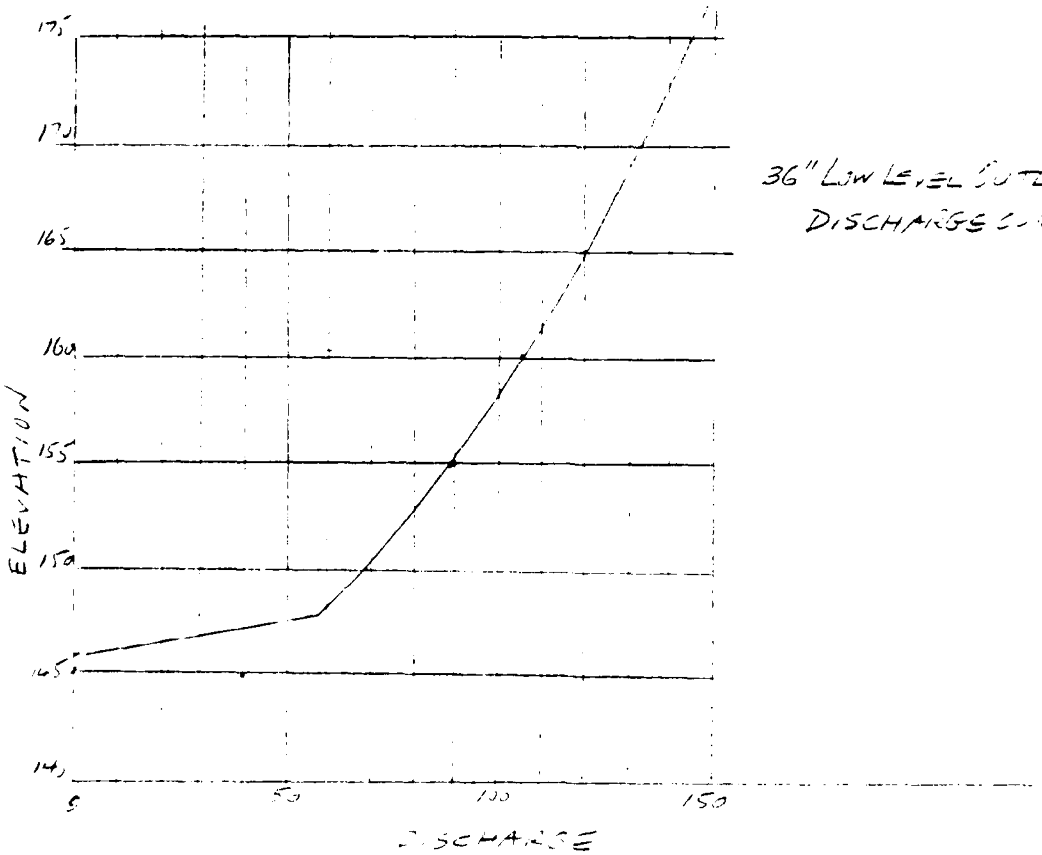
CHKD. BY ______ DATE ______ PROJECT ______

SUBJECT [illegible] POND - OUTLET [illegible] DISCHARGE



36" pipe A = 7.07 sq ft

Σk		El.	Head	Q cfs
[illegible]	0.5			
[illegible] 1.25	3.25			
[illegible] 1.25	2.50	145	2.5	40
[illegible]	2.0	150	7.5	69
[illegible]	1.0	155	12.5	90
K_{valves} [illegible]	[illegible]	160	17.5	106
	5.15	165	22.5	120
Say	5.0	170	27.5	133
		175	32.5	145



36" LOW LEVEL OUTLET DISCHARGE CURVE

D-3

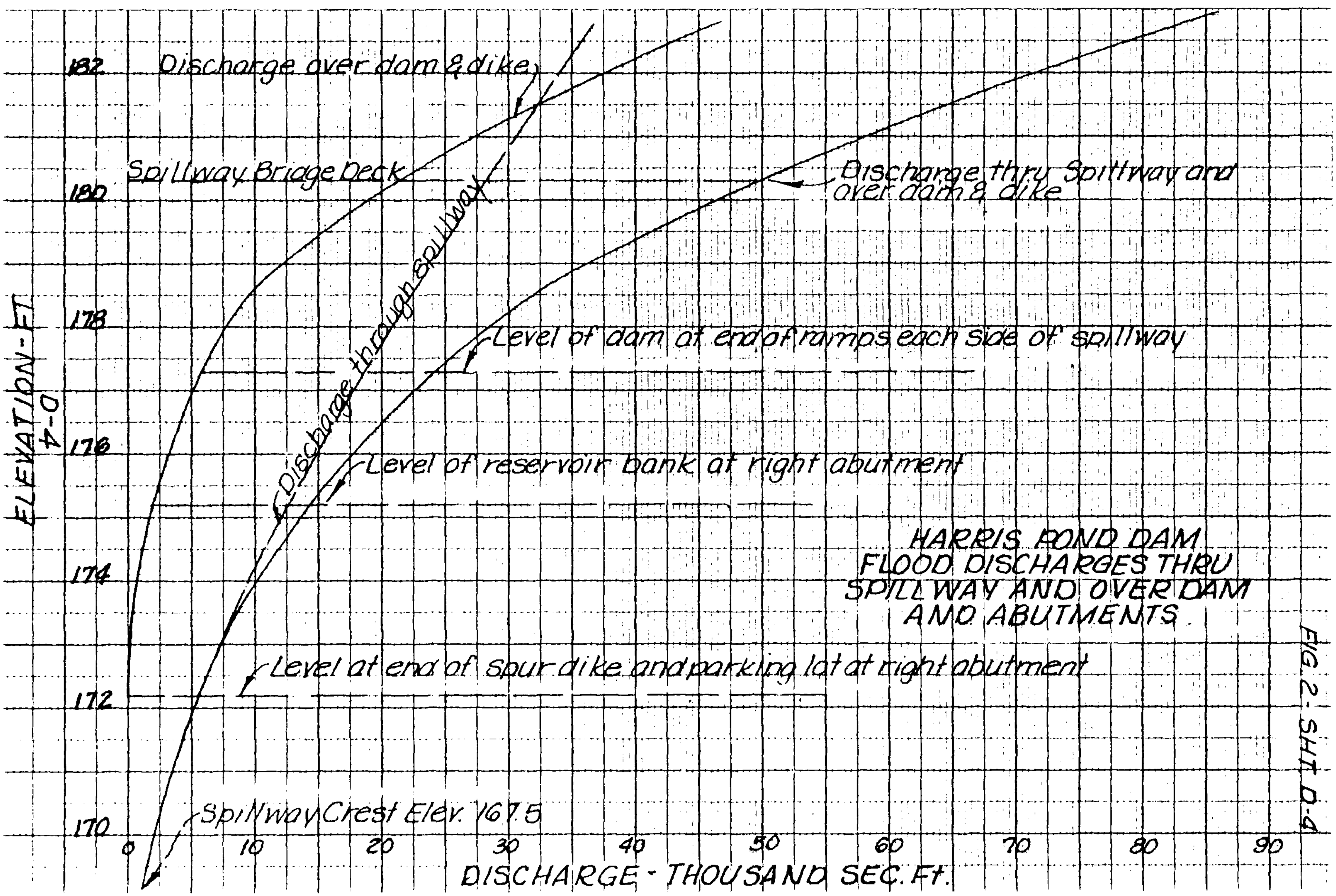


FIG. 2 - SHT. D-4
HARRIS POND DAM
FLOOD DISCHARGES THRU
SPILLWAY AND OVER DAM
AND ABUTMENTS.
Discharge over dam & dike
Spillway Bridge Deck
Discharge thru Spillway and over dam & dike
Discharge through Spillway
Level of dam at end of ramps each side of spillway
Level of reservoir bank at right abutment
Level at end of spur dike and parking lot at right abutment
Spillway Crest Elev. 167.5
ELEVATION - FT
182
180
178
176
174
172
170
0
10
20
30
40
50
60
70
80
90
DISCHARGE - THOUSAND SEC. Ft.

D-4

BY [illegible] DATE 10-9-7[illegible] **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. D-5 OF

CHKD. BY DATE INSPECTION OF DAMS- COAST R.I. PROJECT

SUBJECT HAR[illegible] POND DAM - DISCHARGES THRU S[illegible]WAY + OVER DAM

For shape of crest $H_o = \pm 6.5'$ $P = 6.5'$ $\frac{P}{H_o} = 1.0$

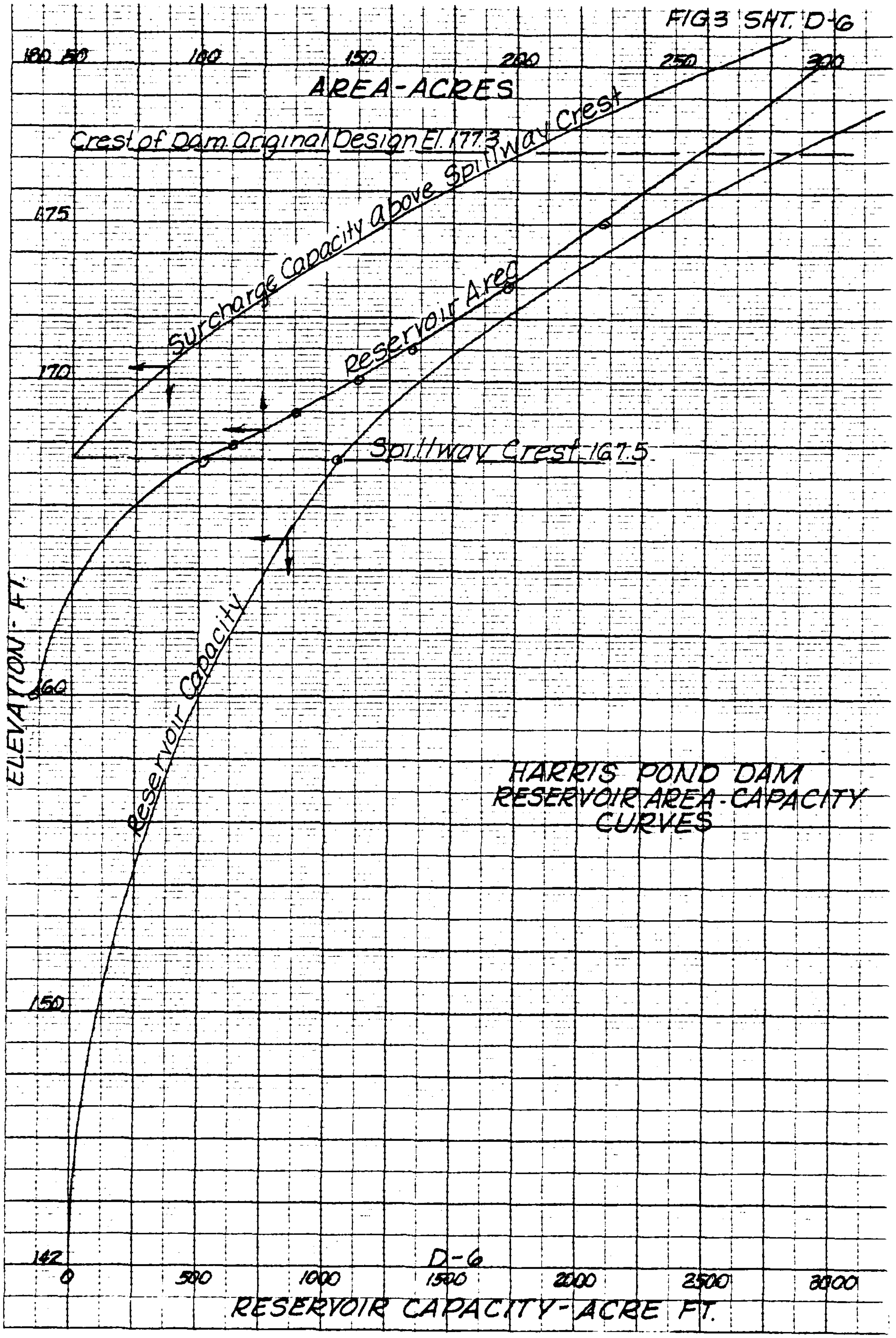
Coeff of sloped crest = 0.99 × coeff for vertical crest

Therefore coeff at design head = 3.88 × 0.99 = 3.84 Say 3.8

	SPILLWAY @ L=150					Right abutment L=80						Spur dike L=300					
Elev.	H	$\frac{H}{H_o}$	$\frac{C}{C_o}$	C	ΔQ	H	C	$\Sigma H^{3/2}$	$H_{av}^{3/2}$	L	ΔQ	H	C	$\Sigma H^{3/2}$	$H_{av}^{3/2}$	L	ΔQ
167.5	0																
168	0.5	0.08	0.81	3.08	163												
169	1.5	0.23	0.86	3.27	901												
170	2.5	0.38	0.895	3.40	2016												
171	3.5	0.54	0.93	3.53	3467												
172	4.7	0.72	0.96	3.65	5579	0	–				0	0	–				0
173.5	6.0	0.92	0.99	3.75	8289	1.3	2.5	3.71	1.86	34.7	65	1.3	2.5	3.71	1.86	70.5	142
175.2	7.7	1.18	1.02	3.87	12403	3.0	2.5	12.99	8.35	80	668	3.0	2.5	12.99	6.50	170.5	1147
177.3	9.8	1.51	1.06	4.02	18545	5.1	2.5	23.77	3.20	80	1456	5.1	2.5	25.79	14.45	300	4320
178.8	11.3	1.74	1.08	4.1	23361	6.6	2.5	42.39	28.69	80	2295	6.6	2.5	42.39	23.46	300	7047
180.3	12.9	1.98	1.1	4.1	28494	8.1	2.5	57.3	43.21	80	3457	8.1	2.5	57.3	35.31	300	10593
181.5	14.0	2.15	1.1	4.1	32215	9.3	2.5	70.9	55.22	80	4204	9.3	2.5	70.4	46.21	300	13863
182.5	15.0	2.3	[illegible]	4.1	35728	10.3	2.5	[illegible]	65.[illegible]	80	5278	10.3	2.5	[illegible]	56.[illegible]	300	18[illegible]42

MAIN DAM RIGHT OF SPILLWAY (180.3 – 660 – 177.3) | DAM LEFT OF SPWY (177.3 – 125 – 180.3; 1100 – 177.3; 177.3)

Elev.	H	C	$\Sigma H^{3/2}$	Ave $H^{3/2}$	L	ΔQ	Ave $H^{3/2}$	L	ΔQ	$H_{av}^{3/2}$	L	ΔQ	Av $H^{3/2}$	L	ΔQ	ΣQ
167.5																
168																163
169																901
170																2016
171																3467
172																5579
173.5																[illegible]
175.2																[illegible]
177.3	–	–				0			0	–		0			0	[illegible]
178.8	1.5	2.7	4.96	2.48	330	818	2.48	65	153	4.9[illegible]	[illegible]	[illegible]	2.4[illegible]	5	12	[illegible]
180.3	3.0	2.8	14.5	7.25	660	4800	7.25	125	9.0	14.5	[illegible]	14.5	7.25	15	[illegible]	[illegible]
181.5	4.2	2.9	24.[illegible]	14.3	660	9[illegible]	14.3	125	17[illegible]	24.[illegible]	[illegible]	24.[illegible]	12.[illegible]	[illegible]	17[illegible]	[illegible]
182.5	5.2	2.9	34.3	21.[illegible]	660	44[illegible]7	21.[illegible]	125	27[illegible]	34.[illegible]	1[illegible]	34.[illegible]	17.[illegible]	[illegible]	2[illegible]	[illegible]



K+E STANDARD CROSS SECTION
10 X 10 TO THE HALF INCH

BY [illegible] DATE [illegible] **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. D-7 OF

CHKD. BY ______ DATE ______ INSPECTION OF DAMS - Conn. R. [illegible] PROJECT ______

SUBJECT [illegible] POND RESERVOIR AREA-CAPACITY CURVES

From U.S.G.S. Quad sheet

Area @ El 168 111.22

170 150.22

180 29[illegible].22

Bottom of Reservoir El. ±142 Storage to Spwy crest El. [illegible]

≈ 1050 AF

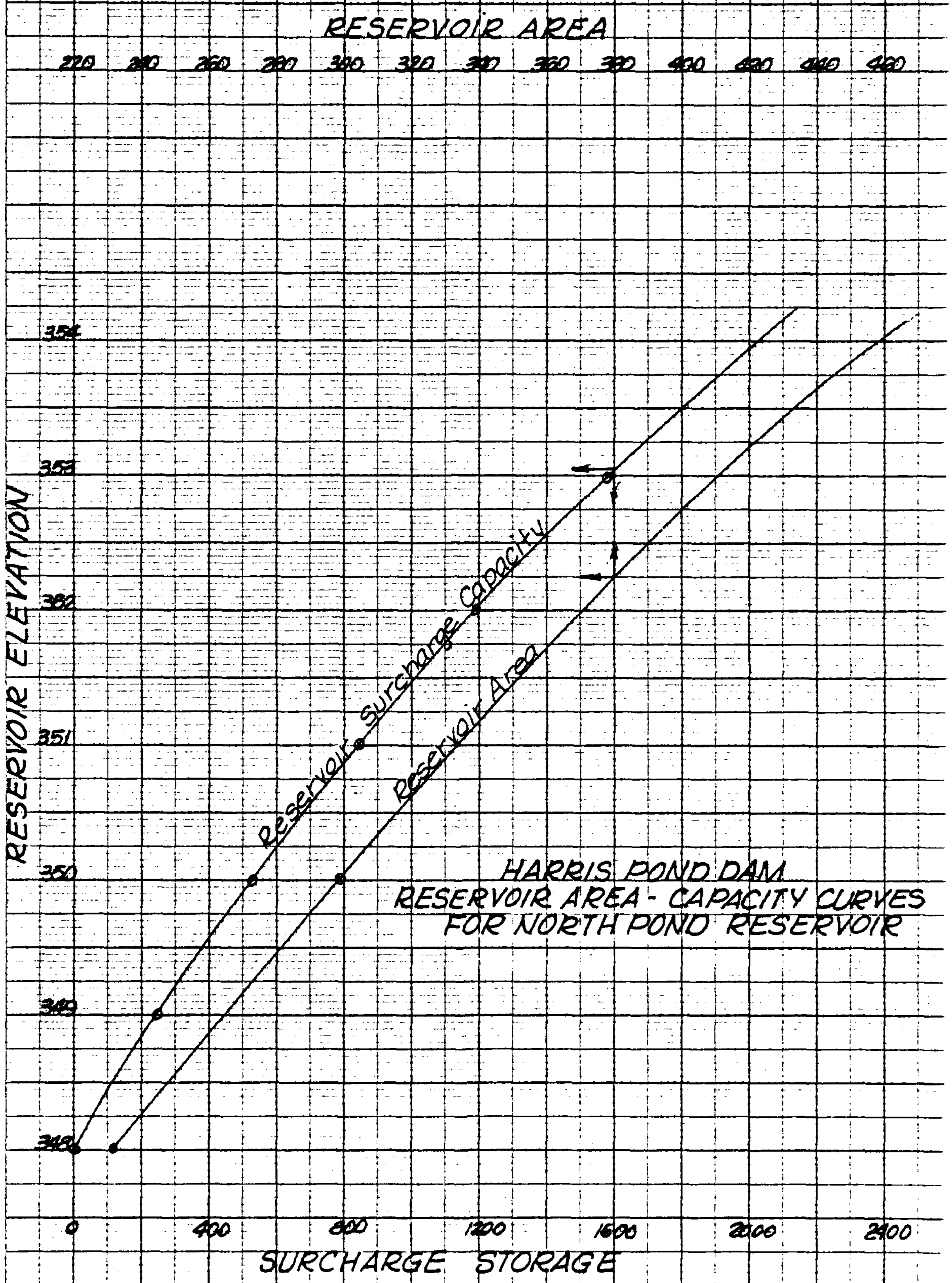
Elev.	Area Acres	Average Area Ac	Δ Stor.	Σ Storage	Surch. Stor.
142	0			0	
150	18	9	72	72	
155	38	28	140	212	
160	55	4[illegible]	240	452	
165	87	72.5	363	815	
167.5	[illegible]	94	235	1050	0
[illegible]	[illegible]	[illegible]	53	1103	53
169	130	121	121	1224	[illegible]74
170	150	140	140	1364	314
171	165	155	155	1522	472
172	179	172	172	1694	644
173	193	187	187	1881	831
[illegible]	[illegible]	[illegible]	2[illegible]	2082	10[illegible]2
175	223	216	[illegible]	2295	[illegible]
176	237	230	230	2525	[illegible]
177	251	244	244	2772	1722
[illegible]	2[illegible]	259	259	3031	[illegible]21
[illegible]	281	273	273	3304	2254
[illegible]	296	289	289	3593	2543

Location	Area Sq. Ft.	Average Depth	Volume Gallons	Evaporation per year in inches	Average yearly rain fall	1" Drawdown from high water in gallons
North Pond	239 acres 10,410,840	10.0'	2396 acre feet 780,813,000	17"	40"	[illegible] 6,506,775
Fiske Mill	20 acres 871,200	3.0'	62.1 acre feet 20,243,743	17"	40"	[illegible] 545,000
West Street	76 acres 3,310,560	1.5'	114 acre feet 37,243,800	17"	40"	[illegible] 2,069,100
Hospital Res. (Freedom Street)	88.4 acres 3,850,704	5.0'	443 acre feet 144,401,400	17"	40"	[illegible] 2,406,690

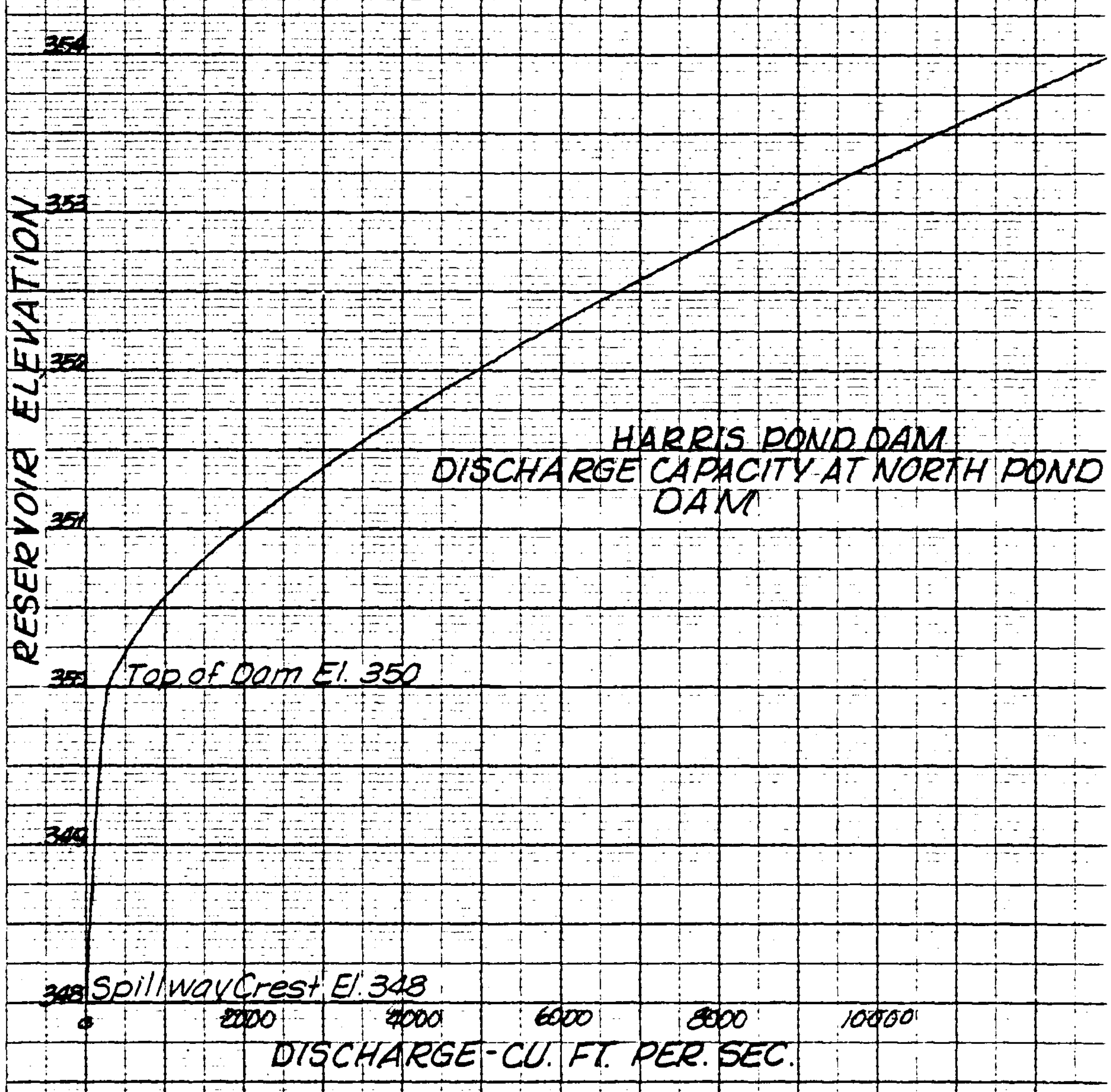
HARRIS POND DAM

UPSTREAM IMPOUNDMENTS ABOVE [illegible] (FREEDOM ST. [illegible])

Source: Draper Division of Rockwell International



KEUFFEL & ESSER CO
MADE IN U.S.A.


HARRIS POND DAM
DISCHARGE CAPACITY AT NORTH POND DAM
RESERVOIR ELEVATION
354
353
352
351
350
349
348
Top of Dam El. 350
Spillway Crest El. 348
0
2000
4000
6000
8000
10000
DISCHARGE-CU. FT. PER. SEC.

BY [illegible] DATE [illegible] **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. 2-11 OF

CHKD. BY DATE INSPECTION OF DAMS - Conn. + R.I. PROJECT

SUBJECT HARR [illegible] POND DAM - North Pond Reservoir - DISCHARGE - STORAGE CURVES

Discharges:

210' 135' 1.0

Top of dam [illegible]

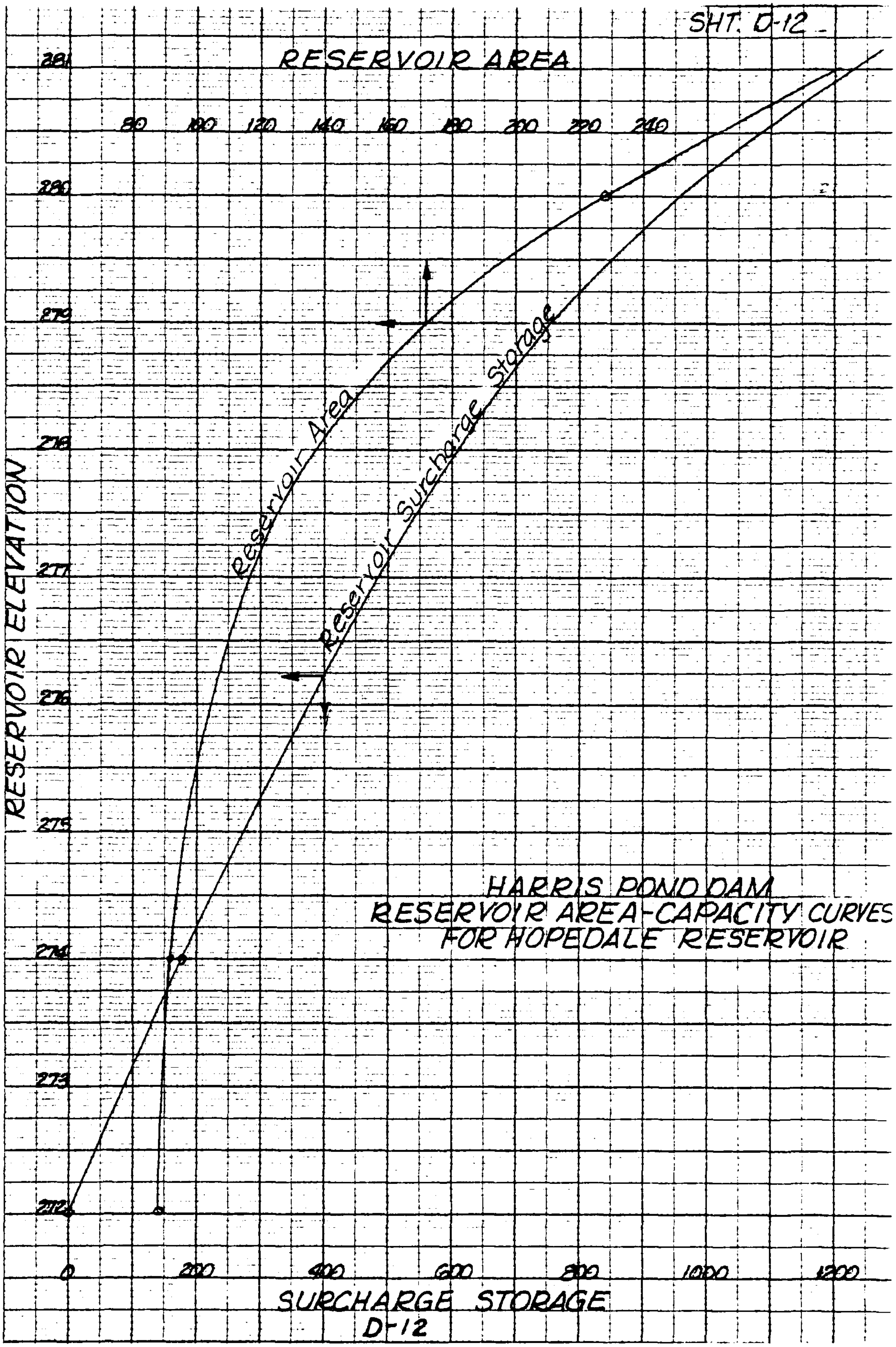
Spillway crest El 348

L = 500

Spillway Elev	C = 2.9 H	Q	Dam C = 3.9 H	Q	Total Q
348	0	0			0
	0.5	33			33
349	1.0	101			101
	1.5	186			186
350	2.0	287	0	0	287
350.5		401	0.5	495	896
351	3	527	1.0	1450	1977
351.5		665	1.5	2664	[illegible]
352	4	812	2.0	4101	[illegible]
352.5		969	2.5	5732	[illegible]
353	5	1135	3.0	7534	8664
353.5		1309	3.5	9494	10803
354	6	[illegible]	4.0	11600	[illegible]

RESERVOIR CAPACITY - SURCHARGE STORAGE

Elev.	Area	Ave Area	Δ Vol.	Σ Vol
348	232		0	0
349	264	248	248	248
350	299	281.5	282	530
351	334	316.5	316	846
352	370	352	352	1198
353	410	390	391	1589
354	459	435	435	2023

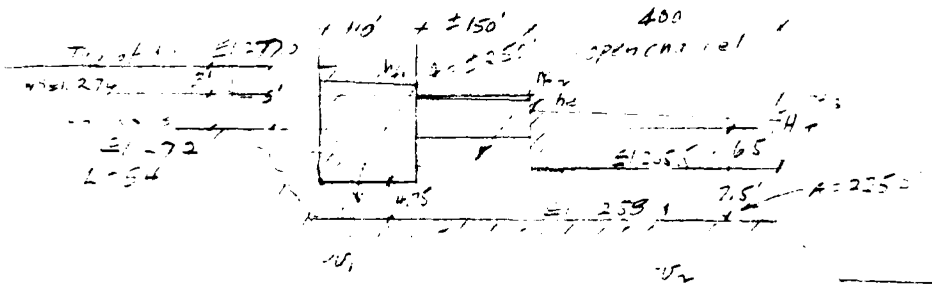

RESERVOIR AREA
80
100
120
140
160
180
200
220
240
281
280
279
278
277
276
275
274
273
272
Reservoir Area
Reservoir Surcharge Storage
RESERVOIR ELEVATION
HARRIS POND DAM
RESERVOIR AREA-CAPACITY CURVES
FOR HOPEDALE RESERVOIR
0
200
400
600
800
1000
1200
SURCHARGE STORAGE

BY ____ DATE 12-1-78 **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. D-13 OF __

CHKD. BY ____ DATE ____ INSPECTION OF DAMS - Conn. & R.I. PROJECT ____

SUBJECT HARRIS POND DAM - Hopedale Res. Capacity & Disch. Curve [illegible]

[illegible] POND DAM - Outlet Capacity [illegible] Spillway/Conduit



Spillway discharge $C = 2.8$ $L = 54$

Elev	Q
274	425
275	780
276	1210
277	1690
278	2222
279	2800
280	3420
281	4080

Q	El.	Summary
3000	8.05	274.35
4000	15.60	281.2
5000		

$A_1 = 4.75 \times 52.6 = 250$ [illegible]

$A_2 = 7.5 \times 30 = 225$ [illegible]

$n = 0.0225$

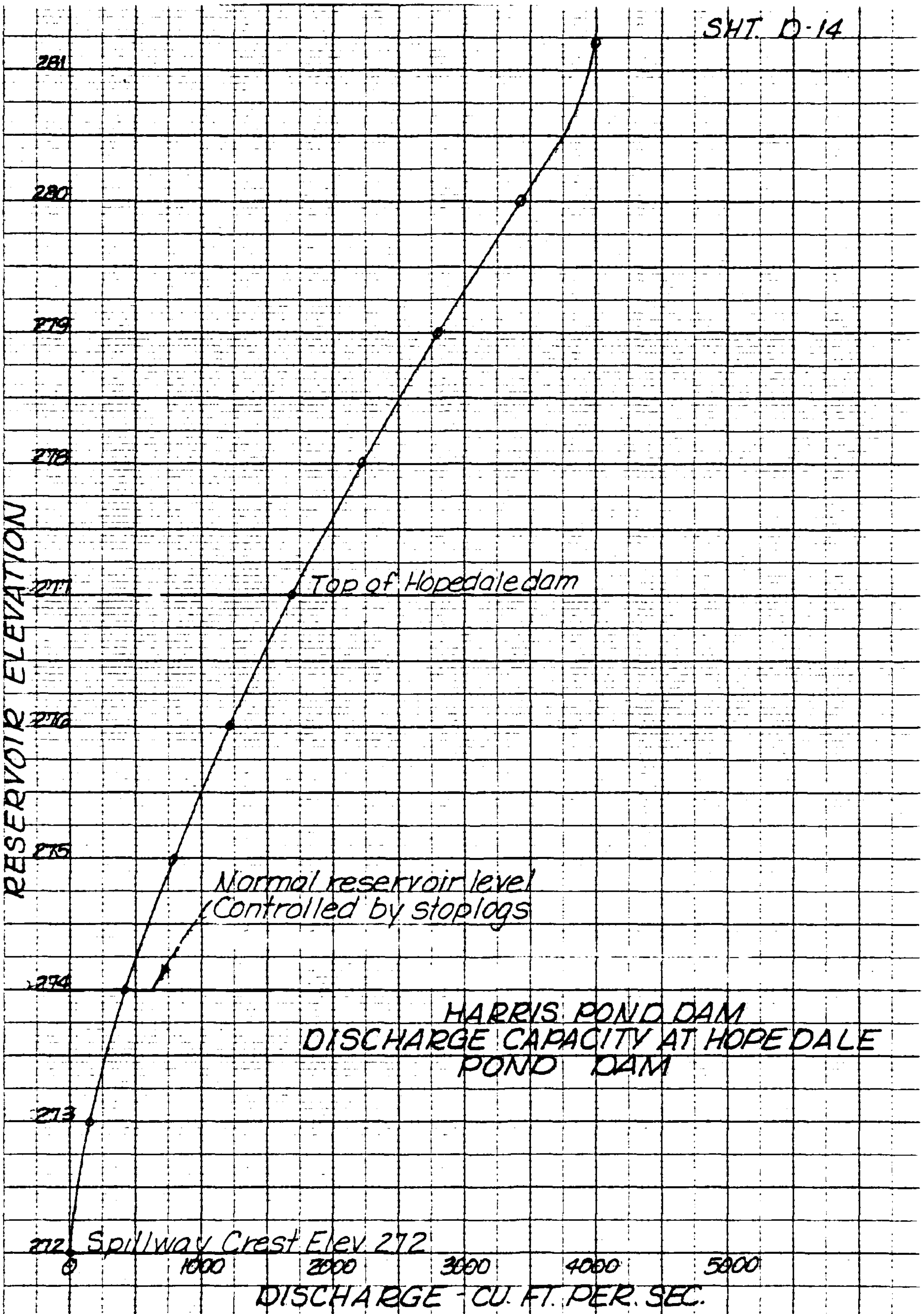
← 52.6 × 4.75 Conduit → 30' × 7.5 conduit

Q	[illegible]	h_e	h_{f1}	h_{e2}	V_3	h_{f3}	h_e	h_{f3}	h_e	[illegible]
3000	12.0	2.24	.22	1.27	0.45	13.33	2.76	0.28	3.77	2.76 [illegible]
4000	16.0	3.98	.40	2.29	0.80	17.78	4.91	0.49	6.70	4.91 [illegible]

RESERVOIR CAPACITY

Elev	Area	A/C Area	Δ vol	Σ vol
272	58.6		0	0
273	90	89	89	89
274	92	91	91	180
275	90	94	94	274
276	104	100	100	374
277	117	110.5	125.5	481
278	132	127.5	127.5	612
279	162	150	150	762
280	220	[illegible]	185	[illegible]
281	300	200	200	[illegible]

D-13

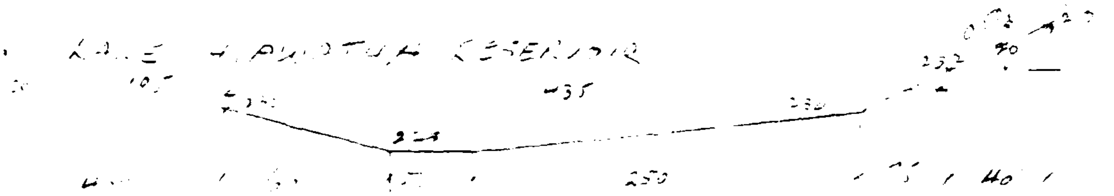


KEUFFEL & ESSER CO
MADE IN U.S.A.

LOUIS BERGER & ASSOCIATES INC.

BY ____________ DATE ________ SHEET NO. D-15 OF __
CHKD. BY ______ DATE ________ INSPECTION OF DAMS - CONN. R.E. PROJECT ____________
SUBJECT [illegible] POND RESERVOIR DAM - LAKE [illegible]

LAKE [illegible] RESERVOIR



Elev	H	C	q/ft	ΔQ	Area ft	L	ΔQ	Avg q/ft	L	ΔQ	ΣQ
229	0	2.8	0								0
230	1	2.8	2.8	140	1.40	310	434	1.4	0	–	574
231	2	2.5	7.72	396	5.30	310	1661	3.70	22	37	[illegible]
232	3	[illegible]	14.55	727	11.24	310	3483	727	40	[illegible]	[illegible]
233	4	2.5	22.4	1120	18.45	310	5728	[illegible]	[illegible]	734	[illegible]
234	5	2.5	31.3	1565	26.85	310	8224	15.65	[illegible]	1377	[illegible]
235	6	2.5	41.16	2055	36.23	310	11231	20.8	[illegible]	2200	[illegible]

RESERVOIR CAPACITY

Elev	Area Acres	Avg Area	Δ Vol	Σ Vol
229	[illegible]			
230	34	32	32	[illegible]
231	38	66	66	128
232	73	70	70	198
233	79	75	75	273
234	83	80	80	353
235	87	84	84	437

BY [illegible] DATE [illegible] **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. D-16 OF

CHKD. BY DATE INSPECTION OF DAMS - CONN + R.I. PROJECT

SUBJECT HARRIS POND RESERVOIRS

WHITE POND RESERVOIR

ELEV	STORAGE	OUTFLOW
348.0	0	0
348.5	130	33
349.0	248	101
349.5	385	186
350.0	530	287
350.5	680	896
351.0	846	1977
351.5	1010	3329
352.0	1198	4913
353.0	1588	8669

HIAWATHA LAKE

ELEV	STORAGE	OUTFLOW
229	0	0
230	62	574
231	28	2140
232	98	4530
233	273	7587
234	353	11156
235	437	15503

HOPEDALE RESERVOIR

ELEV	STORAGE	OUTFLOW
272	0	0
273	89	150
274	180	428
275	276	755
276	374	1210
277	481	1690
278	592	2222
279	702	2800
280	957	3421
281	1221	4000

HARRIS POND RESERVOIR

ELEV	STORAGE	OUTFLOW
167.5	0	0
168.0	55	163
169.0	175	901
170.0	310	2016
171.0	470	3467
172.2	680	5579
173.5	930	5476
175.2	1225	1421
177.5	1830	2432
178.5	2185	3455

BY JH DATE 1-29-79 **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. D-17 OF

CHKD. BY ______ DATE ______ INSPECTION OF DAMS CONN. + R.I. PROJECT ______

SUBJECT [illegible] POND DAM - [illegible]

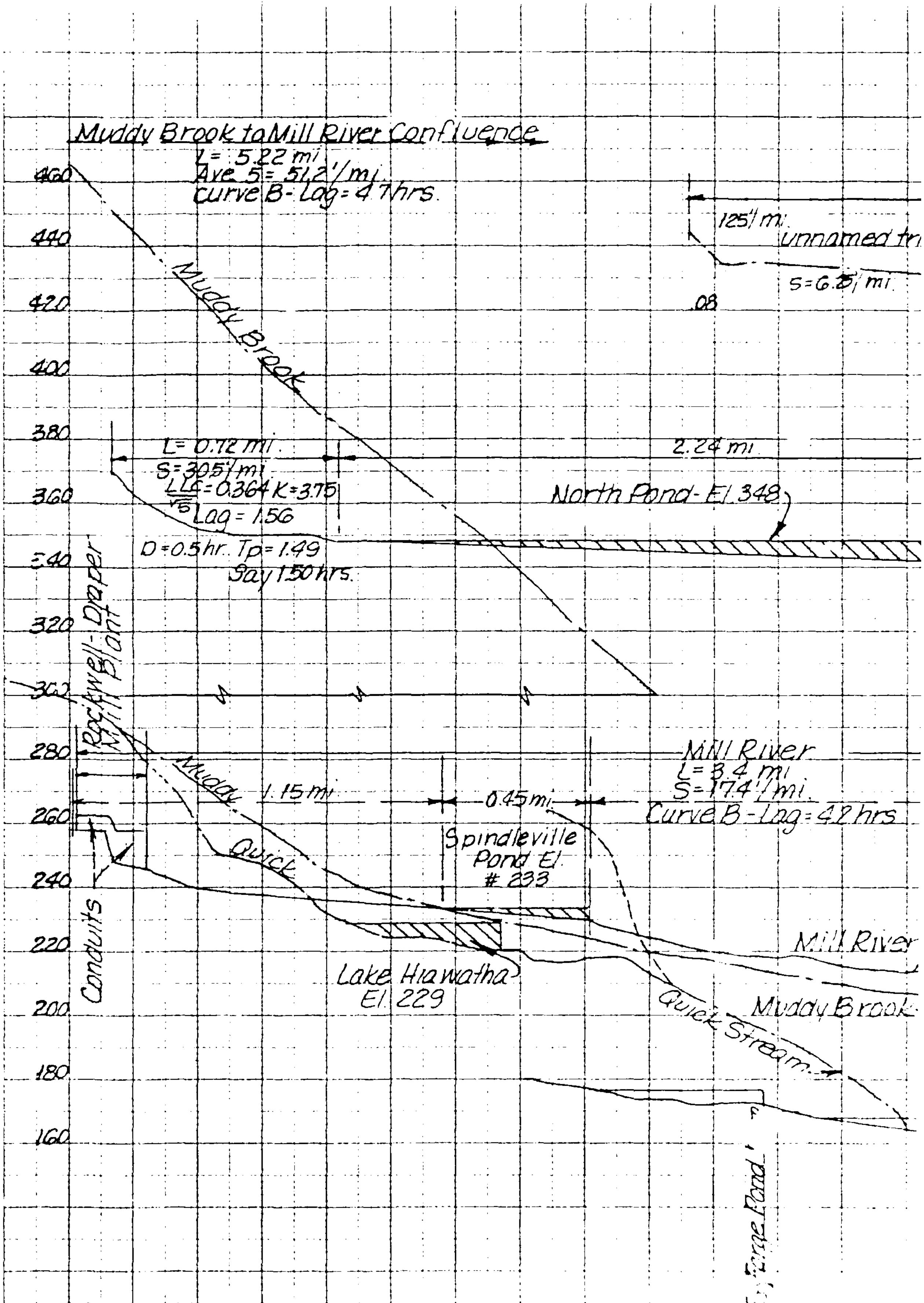
RAINFALL FOR 10 sq mi area 23.5 inches

Reduction factors for 32.5 sq mi = 87%

% of 10 sq mi area for 32.5 sq mi = 88%

∴ Total adjusted rainfall = 87% × 88% = 76.5% × 23.5 = 18"

Time 0.5 hrs	Rearranged % of 6 hr	inches Σ Precip	Infiltration Loss	Rainfall Excess
0.5	5.0	0.90	0.5	0.40
1.0	5.0	0.90	0.5	0.40
1.5	5.5	0.99	0.1	0.89
2.0	6.5	1.17	0.1	1.07
2.5	7.0	1.26	0.1	1.16
3.0	8.0	1.44	0.1	1.34
3.5	10.0	1.80	0.1	1.70
4.0	28.0	5.04	0.1	4.94
4.5	7.0	1.26	0.1	1.16
5.0	7.0	1.26	0.1	1.16
5.5	6.0	1.08	0.1	0.98
6.0	5.0	0.90	0.1	0.80
		18.00	2.0	16.00


Muddy Brook to Mill River Confluence
L = 5.22 mi
Ave S = 51.2'/mi
curve B - Lag = 4.7 hrs.
460
440
420
400
380
360
340
320
300
280
260
240
220
200
180
160
Muddy Brook
125'/mi
unnamed tri
S = 6.25'/mi
.08
L = 0.72 mi
S = 305'/mi
LLc/√S = 0.364 K = 3.75
Lag = 1.56
D = 0.5 hr. Tp = 1.49
Say 1.50 hrs.
2.24 mi
North Pond - El. 348
Rockwell-Draper Mill Plant
Muddy
Quick
1.15 mi
0.45 mi
Mill River
L = 3.4 mi
S = 17.4'/mi
Curve B - Lag = 4.2 hrs
Spindleville Pond El. # 233
Lake Hiawatha El. 229
Conduits
Mill River
Muddy Brook
Quick Stream
Pond

AD-A156 587 NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS HARRIS POND DAM (RI 0..(U) CORPS OF ENGINEERS WALTHAM MA NEW ENGLAND DIV FEB 79 2/2

UNCLASSIFIED F/G 13/13 NL

END
DATE
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8-85
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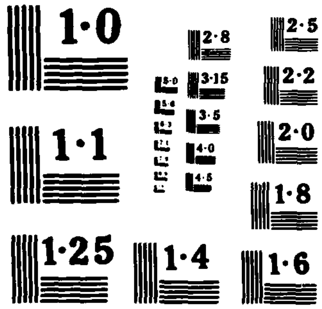
1·0
2·8
2·5
3·15
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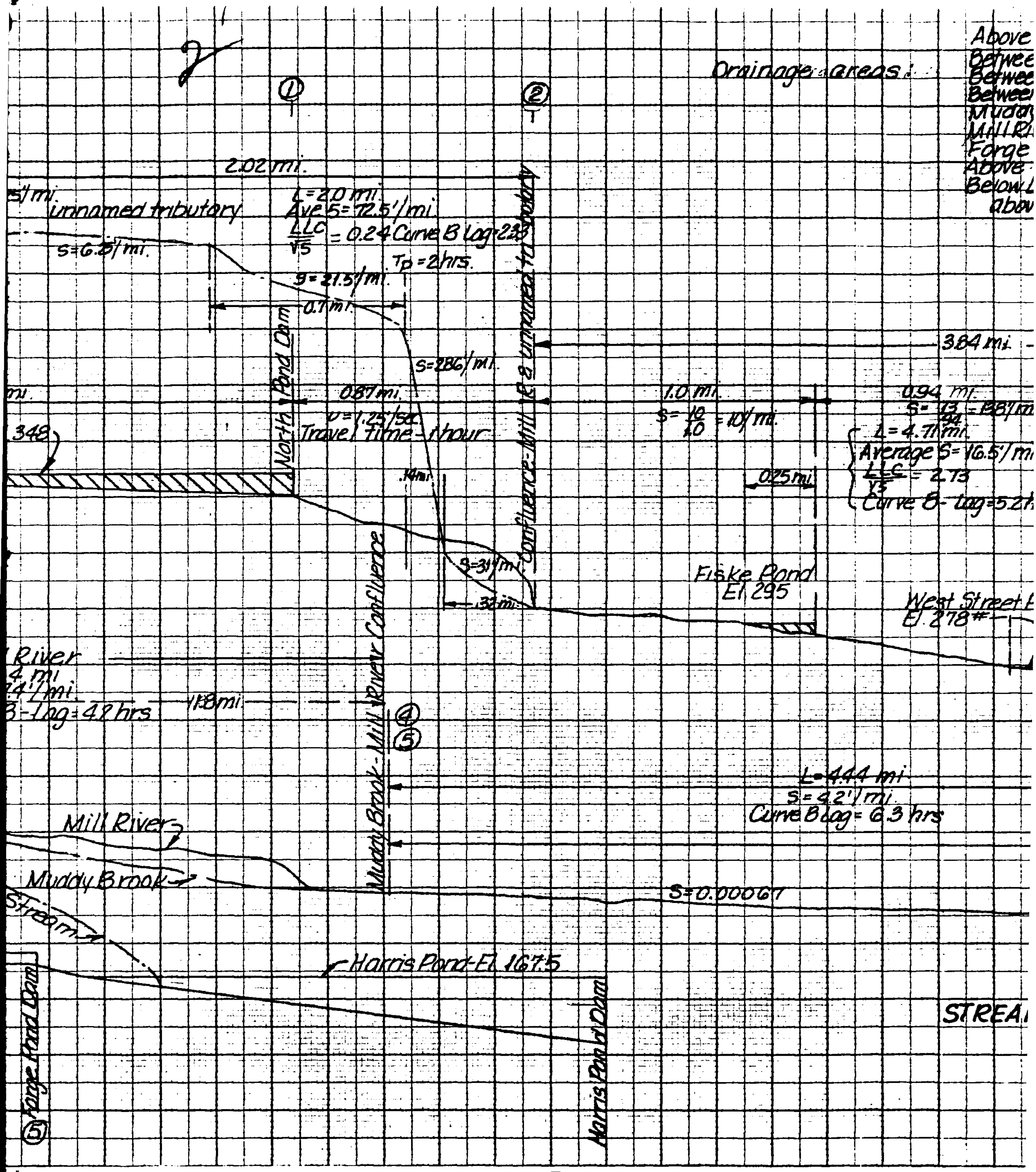
1·6

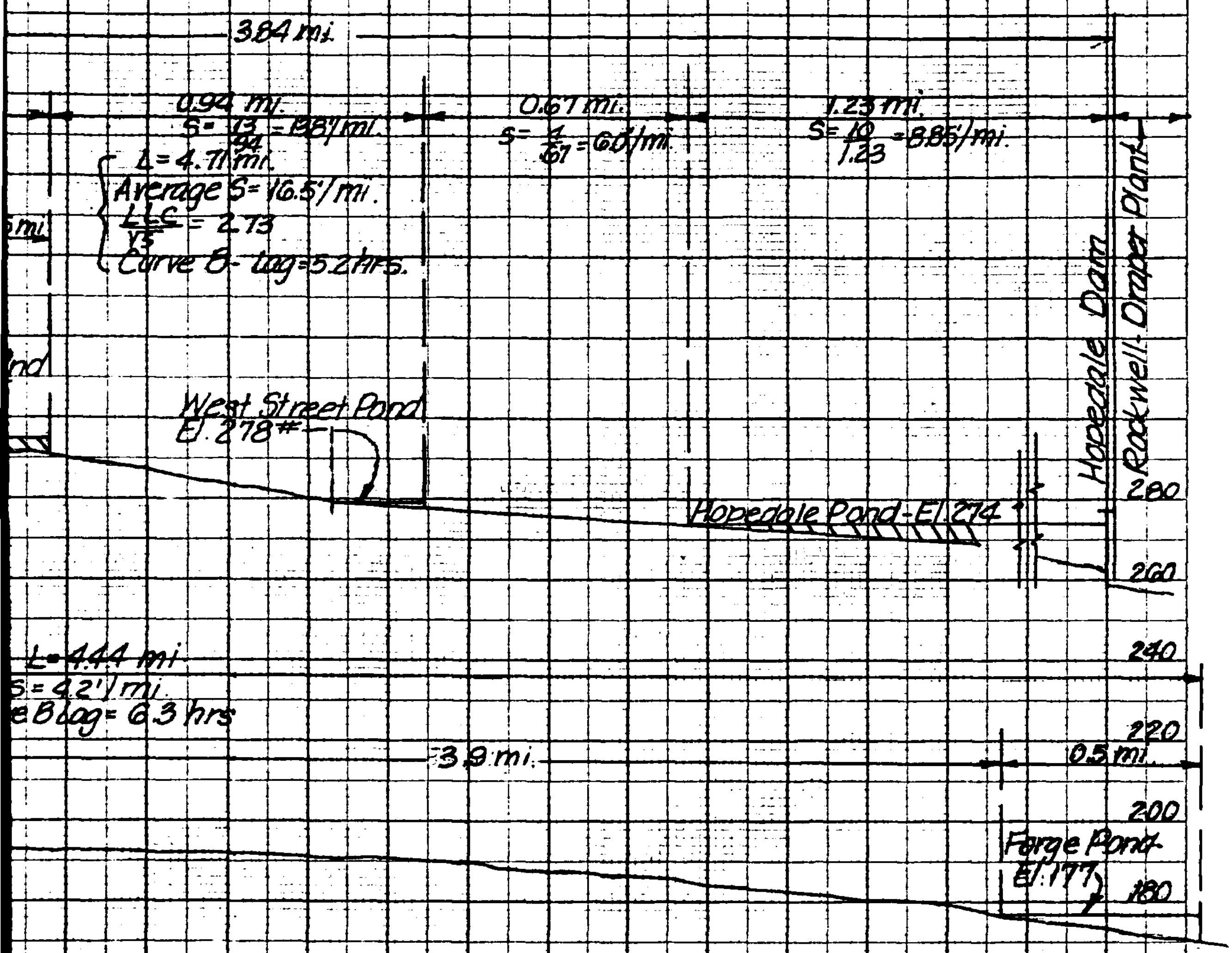


7
Drainage areas:
Above
Betwee
Betwee
Betwee
Mudd
Mill Ri
Forge
Above
Below L
abov
①
②
2.02 mi.
5'/mi.
unnamed tributary
L=2.0 mi.
Ave S=72.5'/mi.
LLC/√S = 0.24 Curve B Lag=2.3
s=6.5'/mi.
Tp=2 hrs.
S=21.5'/mi.
0.7 mi.
North Pond Dam
S=286'/mi.
Confluence - Mill R. & unnamed tributary
3.84 mi.
mi.
0.87 mi.
v=1.25'/sec.
Travel time-1 hour
1.0 mi.
S=10/1.0=10'/mi.
0.94 mi.
S=13/.94=13.8'/mi
L=4.71 mi.
Average S=16.5'/mi
LLC/√S = 2.73
Curve B-Lag=5.2 h
348
.14 mi.
0.25 mi.
S=31'/mi.
Fiske Pond
El. 295
.32 mi.
West Street
El. 278±
Muddy Brook-Mill River Confluence
River
4 mi.
4'/mi.
B-Lag=4.8 hrs
1.18 mi.
④
⑤
L=4.44 mi.
S=4.2'/mi.
Curve B Lag=6.3 hrs
Mill River
Muddy Brook
S=0.00067
Stream
Harris Pond-El. 167.5
Forge Pond Dam
⑤
Harris Pond Dam
STREA

ge areas:

Above North Pond Dam	2.7 sq.mi.	
Between unnamed Brook confluence & North P. Dam	1.7 sq.mi.	
Between Hopeville Dam & unnamed Brook Confluence	5.6 sq.mi.	
Between Muddy Brook Confluence & Hopeville Dam	2.8 sq.mi.	
Muddy Brook above Mill River confluenec	6.1 sq.mi.	
Mill River Below Muddy Confluence above Forge Pond Dam	9.4 sq.mi.	
Above Lake Hiawatha Dam	1.1 sq.mi.	
Below Lake Hiawatha & Forge Pond Dam & above Harris Pond dam	3.1 sq.mi.	
	Total	32.5 sq.mi.

(3)



HARRIS POND DAM
STREAM PROFILES ABOVE HARRIS POND RES.

BY [illegible] DATE 1-24-7[illegible]

LOUIS BERGER & ASSOCIATES INC.

SHEET NO. D-19 OF

CHKD. BY DATE

PROJECT

SUBJECT HARRIS POND DAM - HYDROLOGY - DEVELOPMENT OF UNITGRAPHS

$$Lag = K\left(\frac{L \times 0.5L}{\sqrt{S}}\right)^{.33} \qquad T_p = 0.75\,Lag + .375D \qquad T_c = 1.67T_p - 0.83D$$

Reach		Longest Stream length - mi Sub	Longest Stream length - mi Total	Stream slope Ft/mi Sub	Stream slope Ft/mi Total	Lag-hrs Curve B K=3.75 Sub	Lag-hrs Total	Computed Unit graph Time to Peak Tp-hrs D=1.0hr	Adopted Tp hrs	Tc hrs	Ave Velocity Ft/sec	Transport Time From-to	Miles	Ave. Velocity ft/sec	Time-hrs
DA = 2.70 sq mi North Branch above (1)			0.72		30.5		1.37	1.75	2.0	2.5	0.42	(1)-(2)	0.87	1.28	1.0
Unnamed Brook		0.08		125.0		0.23									
Above (2)		0.78		6.25		1.85									
DA = 1.70 sq mi		0.70		21.5		1.43									
		0.14		286		0.33									
		0.32		310		0.80									
			2.02		72.5		4.26	3.37	4.0	5.85	0.51	-	-	-	-
Hopedale Dam to	Fisk Pond	1.0		10.0		2.04									
Unnamed Brook	West St. Pond	0.44		13.8		1.86									
(2) to (3)	Hopedale Res.	0.67	2.61	6.0		1.70	5.6								
DA = 5.6 sq mi	Hopedale Dam	1.23		8.85											
			3.84		9.86		4.47	4.10	4.0	5.85	0.65	(2)-(3)	2.61	1.27	3.0
Mill River Meeting [illegible]	Rakwell Mill	0.40		85.0		-									0.10
[illegible] Pond	Spindleville Pond	0.75		8.3		1.24									
(3) to (4)	Spindleville Dam	0.45		8.3		1.24									
DA = 2.80 sq mi	[illegible]	1.40		14.3		2.40									
	[illegible]	0.10		1000		0.30									
	[illegible]	0.30	3.40	6.7		1.00	[illegible]								
			3.00		11.3		4.07	[illegible]	5.0	7.5	0.67	(3)-(4)	3.4	1.0	5.0
Mill Brook above Mill River confluence (4)		2.30		83.3		2.77									
		2.0		20.		2.57									
DA = 1.14 sq mi		1.52		3.3		1.55	[illegible]								
			5.12		51.1		4.64	[illegible]	5.0	7.5	1.0	-	-	-	-

D-19

BY: [illegible] DATE: 1-29-79

LOUIS BERGER & ASSOCIATES INC.

SHEET NO. D-20 OF

CHKD. BY ______ DATE ______

PROJECT ______

SUBJECT: HARRIS POND DAM - HYDROLOGY - DEVELOPMENT OF UNITGRAPHS

REACH		Longest stream length-miles: Sub	Longest stream length-miles: Total	Stream slope ft/mi: Sub	Stream slope ft/mi: Total	Lag time hrs Curve B k=3.75: Sub	Lag time hrs Curve B k=3.75: Total	Computed Unitgraph Time to peak Tp in hrs D=1	Adopted Tp hrs	Tc hrs	Average velocity ft/sec	Transport time: From-To	Transport time: Miles	Transport time: Ave Velocity ft/sec	Transport time: Time hrs
Mudding Mill confluence to Forge Pond Dam	To Forge Pond	3.9		5.38		5.55									
	To Forge Dam	0.5		-			.35	4.50							
DA = 9.4 mi.			4.4		5.96		5.90	4.50	5.0	7.5	0.86	(5)-(6)	4.4	1.29	5.0
Above Lake Hiawatha Dam. (8)	Quick R.														
DA = 1.1 sq. mi.			1.23		65.1		1.71	1.66	2.0	2.5	0.72	(8)-(7)	1.26	1.23	1.5
Between Harris Pond Dam and Forge Pond Below (6) (8) and (7)	Quick Brook to Dam														
DA = 3.1 sq mi.			1.10		88.2		1.52	1.51	1.5	1.7	0.95	-	-	-	-
Total			21.53						Weighted average		0.75		12.50	1.2 ← Weighted average	

D-20

BY: ___ DATE: 1-29-79 **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. D-21 OF ___

CHKD. BY ___ DATE ___ INSPECTION OF DAMS - CONN. R.I. PROJECT ___

SUBJECT: HARRIS POND DAM - HYDROLOGY

CURVILINEAR UNITGRAPHS

Time hrs	T/T_p (T_p = 4.0)	Q/Q_p	Q_p=205.7 Q	Q_p=677.6 Q	T/T_p (T_p = 5.0)	Q/Q_p	Q_p=271.04 Q	Q_p=590.48 Q	Q_p=909.72 Q
0.5	.125	.030	6	20	0.1	0.015	4	9	14
1.0	0.25	.116	24	80	0.2	0.075	20	44	68
1.5	0.375	.250	51	169	0.3	0.16	43	94	146
2.0	0.50	.430	88	291	0.4	0.28	76	165	255
2.5	0.625	.642	132	435	0.5	0.43	117	254	391
3.0	0.75	.830	170	562	0.6	0.60	163	354	546
3.5	0.875	.950	195	644	0.7	0.77	209	455	701
4.0	1.00	1.000	206	678	0.8	0.89	241	526	810
4.5	1.125	.965	199	654	0.9	0.97	263	573	883
5.0	1.25	.880	181	596	1.0	1.00	271	590	910
5.5	1.375	.773	159	524	1.1	0.98	266	579	892
6.0	1.50	.660	136	447	1.2	0.92	249	543	837
6.5	1.625	.543	112	368	1.3	0.84	228	495	764
7.0	1.75	.455	94	308	1.4	0.75	203	443	682
7.5	1.875	.382	79	259	1.5	0.66	179	390	601
8.0	2.00	.320	66	217	1.6	0.56	152	331	510
8.5	2.125	.270	56	183	1.7	0.49	133	289	446
9.0	2.25	.225	46	152	1.8	0.42	114	248	382
9.5	2.375	.187	38	127	1.9	0.37	100	218	337
10.0	2.50	.155	32	105	2.0	0.32	87	189	291
10.5	2.625	.126	26	85	2.1	0.28	76	165	255
11.0	2.75	.106	22	72	2.2	0.24	65	142	218
11.5	2.875	.090	19	61	2.3	0.21	57	124	191
12.0	3.00	.075	15	51	2.4	0.18	49	106	164
12.5	3.125	.0652	13	44	2.5	0.155	42	92	141
13.0	3.25	.0555	11	38	2.6	0.13	35	77	118
13.5	3.375	.0457	9	31	2.7	0.114	31	67	104
14.0	3.50	.035	7	24	2.8	0.099	27	58	89
						.0565	23	51	79
							20	44	68
							18	40	61
							16	35	54
							14	30	47
							12	26	41

$T_p = 4.0$, DA = 1.7, Q_p/... = $\frac{484 \times A}{T_p}$ = 205.7

$T_p = 4.0$, DA = 5.6, Q_p " = 677.6

$T_p = 5.0$, DA = 2.8, Q_p = $\frac{484 \times A}{5}$ = 271.04

$T_p = 5.0$, DA = 6.1, Q_p = 590.48

$T_p = 5.0$, DA = 9.4, Q_p " = 909.72

BY SJH DATE 1-29-79 **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. D-22 OF

CHKD. BY DATE INSPECTION OF DAMS - CONN T R I PROJECT

SUBJECT HARRIS POND DAM - HYDROLOGY

CURVILINEAR UNITGRAPHS

TIME HRS	$T_p=1.5$ T/Tp	Q/Qp	Qp=1000 Q	$T_p=2.0$ T/Tp	Q/Qp	Qp=266.2 Q	Qp=653.4 Q
0.5	0.33	0.19	190	0.25	0.115	31	75
1.0	.667	0.72	720	0.50	.43	114	281
1.5	1.00	1.00	1000	0.75	.84	224	549
2.0	1.33	0.81	810	1.00	1.00	266	653
2.5	1.67	0.51	510	1.25	.88	234	575
3.0	2.00	0.32	320	1.50	.66	176	431
3.5	2.33	0.20	200	1.75	.45	120	294
4.0	2.67	0.115	115	2.00	0.32	85	209
4.5	3.00	.075	75	2.25	.225	60	147
5.0	3.33	.045	45	2.50	.153	41	100
5.5	3.67	.03	30	2.75	.104	28	68
6.0	4.00	.018	18	3.00	.075	20	49
6.5	4.33	.011	11	3.25	.052	14	34
7.0	4.67	.005	5	3.50	.035	9	23
7.5	5.00	.002	2	3.75	.025	7	16
8.0	5.33			4.00	.018	5	12
8.5	5.67			4.25	.013	3	8
9.0	6.00			4.50	.009	2	6
9.5	6.33			4.75	.005	1	3
10.0	6.67			5.00	.002		1
10.5	7.00			5.25			
11.0	7.33			5.50			
11.5	7.67			5.75			
12.0	8.00			6.00			

$T_p = 1.5$ $A = 3.10$ sq mi Q_p/inch $= \frac{484 \times A}{T_p} = 1000$

$T_p = 2.0$ $A = 1.1$ sq mi. Q_p/inch $= \quad = 266.2$

$T_p = 2.0$ $A = 2.70$ sq mi Q_p/inch $= \quad = 653.4$ Above Pt. 1

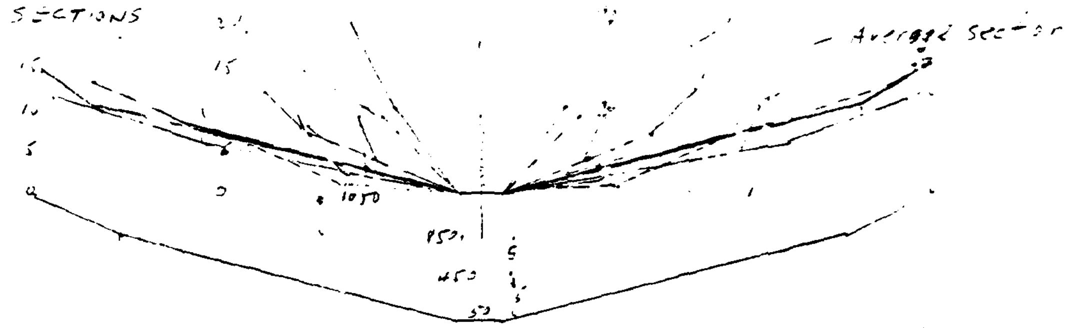
BY D.J.K. DATE 11/29/78 **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. D-23 OF

CHKD. BY ______ DATE ______ INSPECTION OF DAMS - CONN. & R.I. PROJECT ______

SUBJECT MILL RIVER - LAG TIME FOR CHANNEL UPSTREAM FROM HARRIS POND

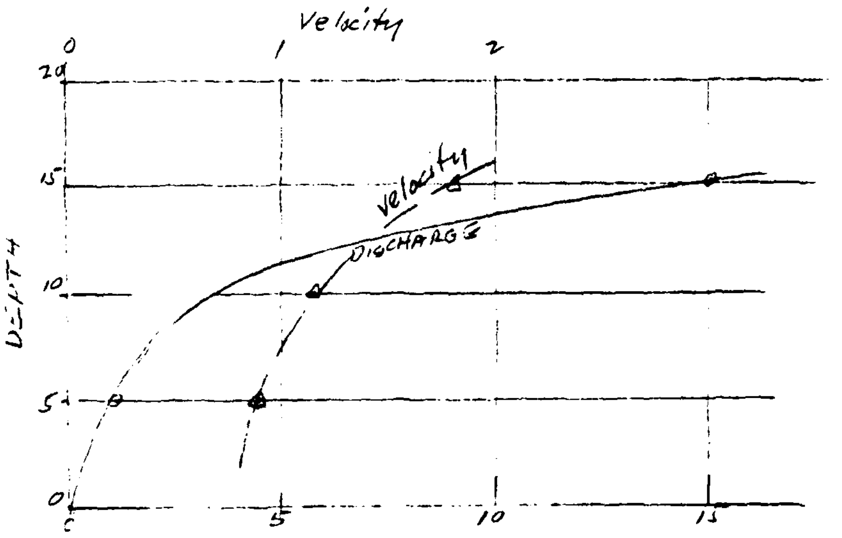
MILL RIVER UPSTREAM FROM FORGE POND

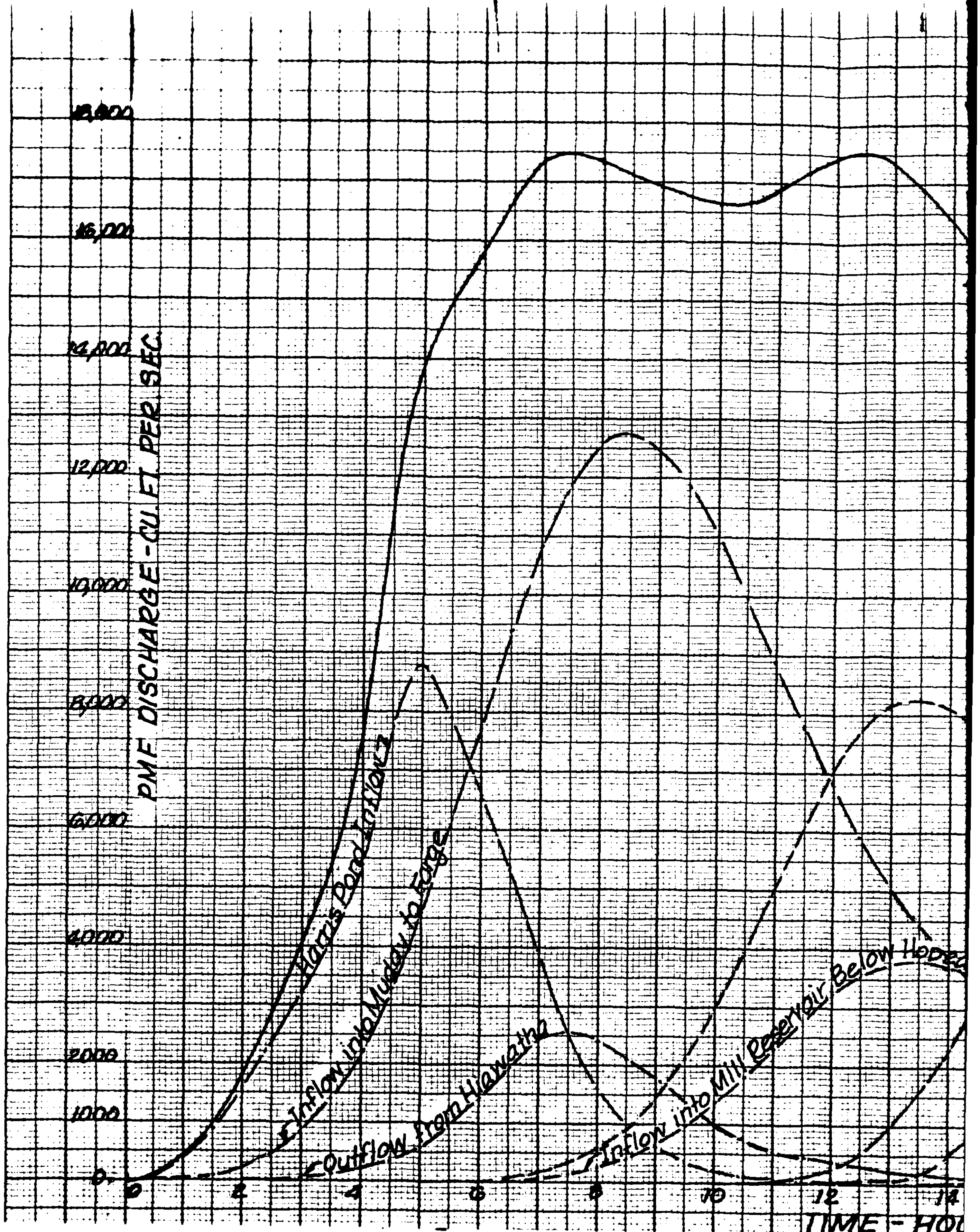
AT RIVER BETWEEN EL 150 and 170 L = 20800 S = .00096 $S^{1/2}$ = .031



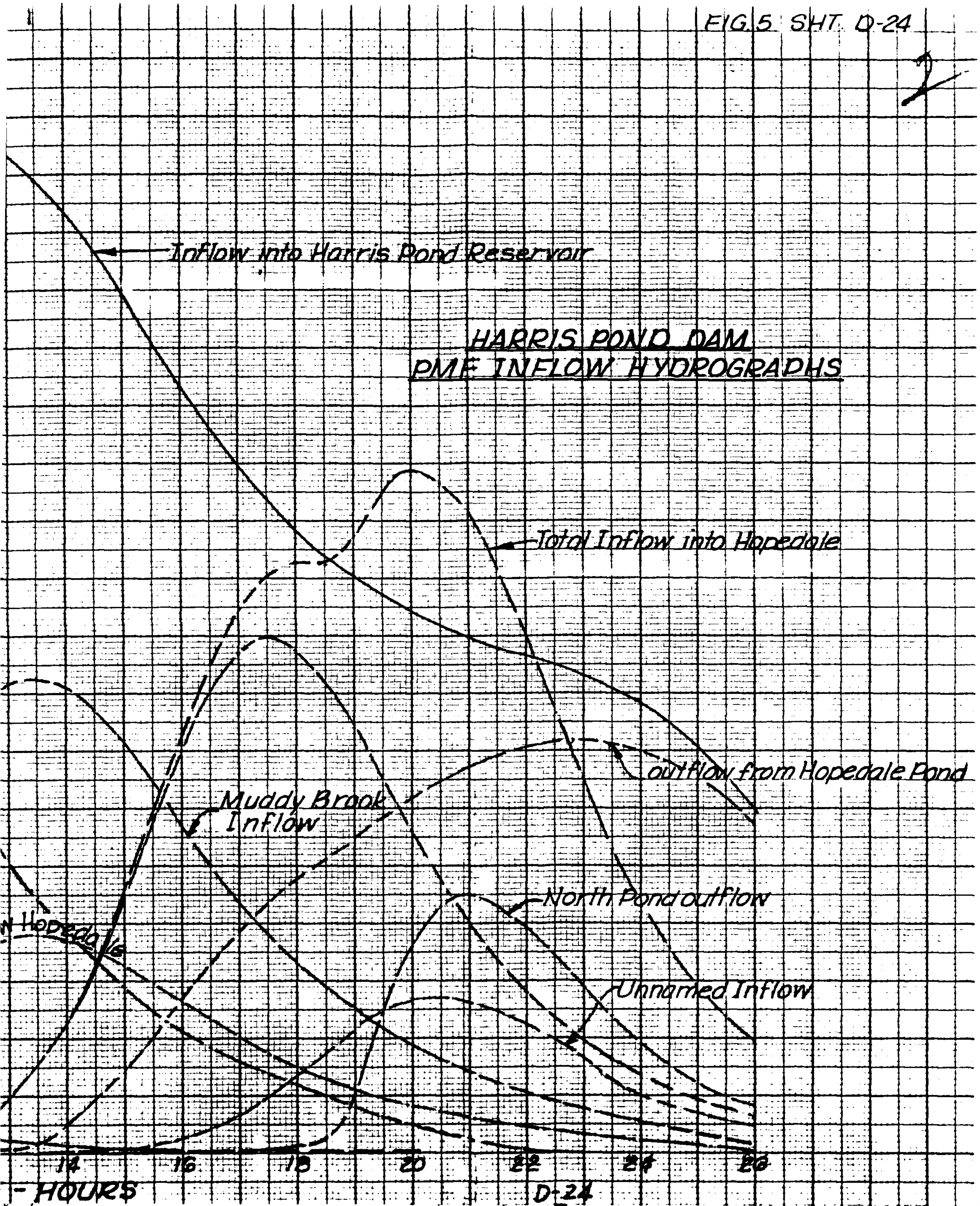
Average S = 0.00096 S = 0.031 n = 0.10 $v = \frac{1.486}{n} r^{2/3} S^{1/2}$

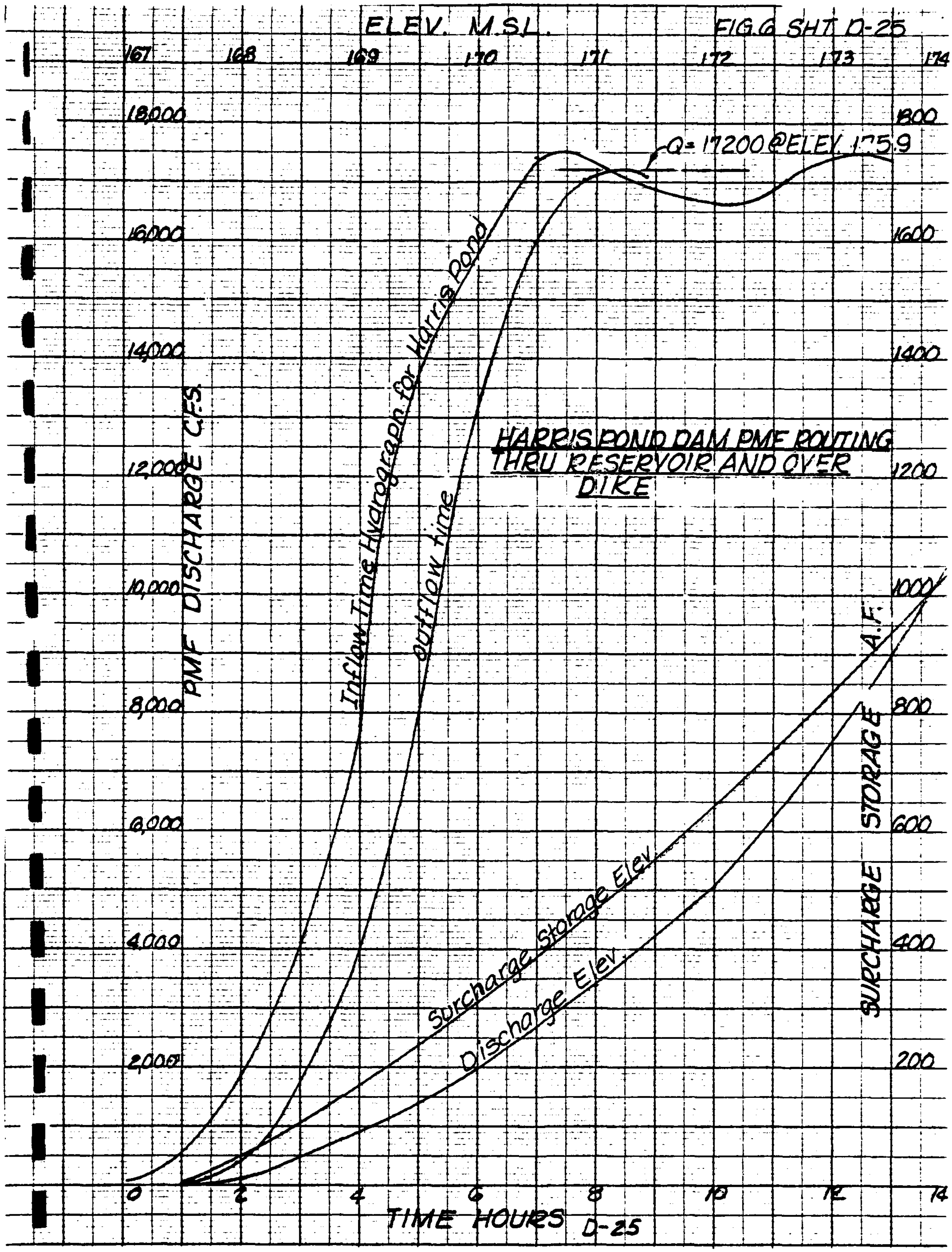
Depth	Δ Area	Σ Area	wp	r	$r^{2/3}$	v	Q cfs
0	0	0					
5	1250	1250	450.1	2.78	1.98	0.91	1138
10	2250	3500	850.2	4.12	2.57	1.18	2632
15	4750	8250	1050.5	7.85	3.95	1.82	15017



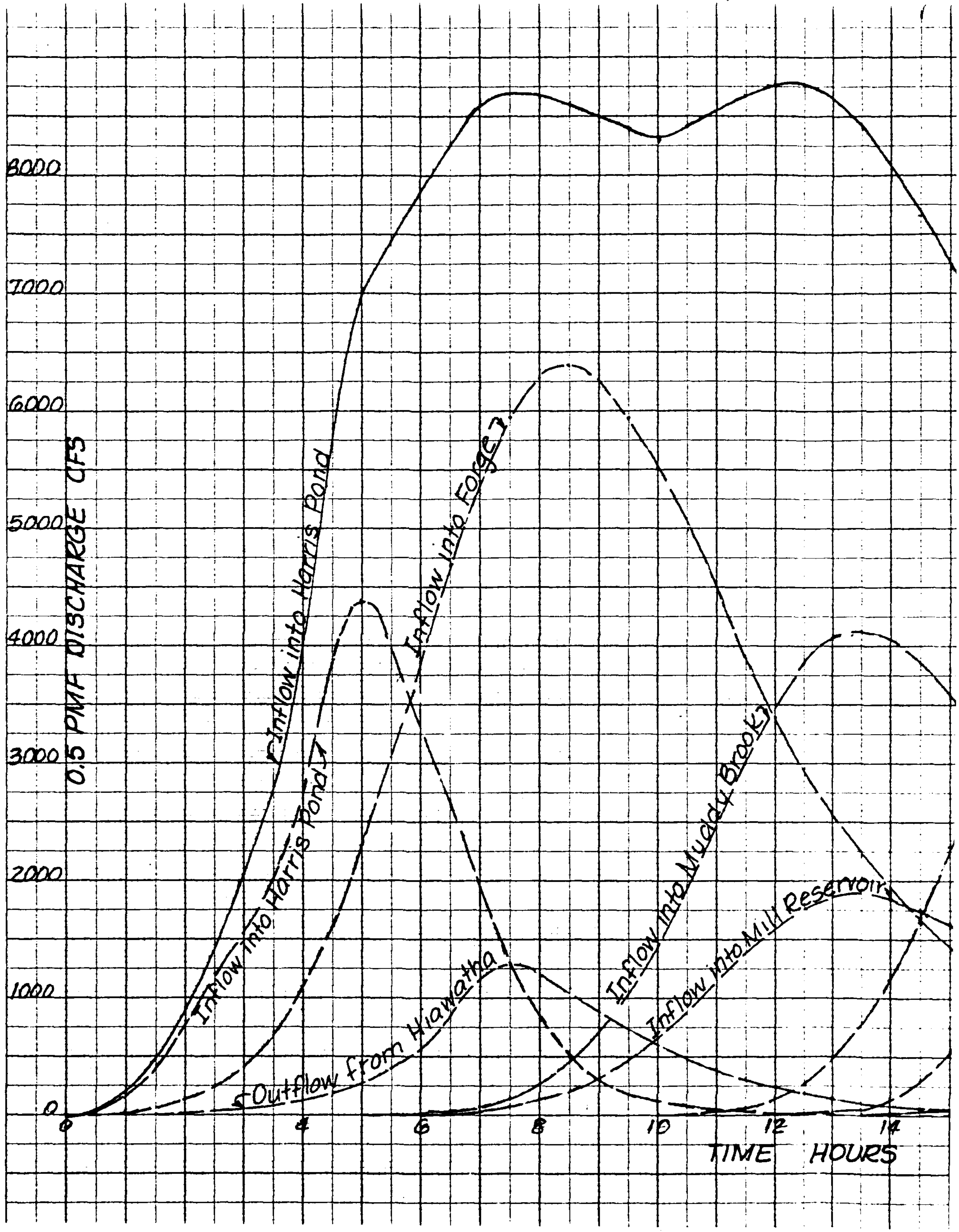


PMF DISCHARGE-CU.FT. PER SEC
18,000
16,000
14,000
12,000
10,000
8,000
6,000
4000
2000
1000
0
0
2
4
6
8
10
12
14
TIME -
Harris Pond Inflow
Inflow into Muddy to Forge
Outflow From Hiawatha
Inflow into Mill Reservoir Below

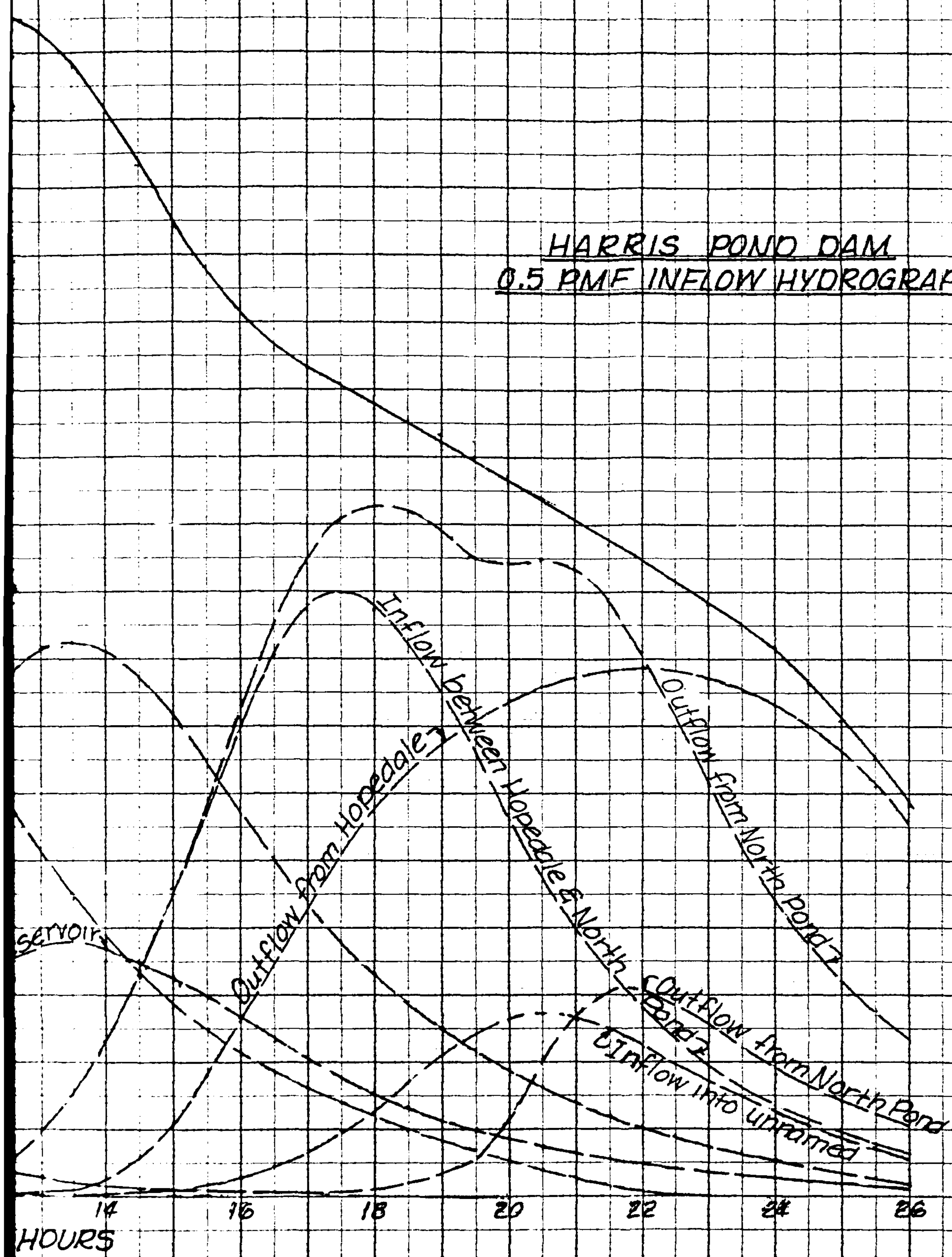

Inflow into Harris Pond Reservoir
HARRIS POND DAM
PMF INFLOW HYDROGRAPHS
Total Inflow into Hopedale
outflow from Hopedale Pond
Muddy Brook Inflow
North Pond outflow
Hopedale
Unnamed Inflow
14
16
18
20
22
24
26
- HOURS



ELEV. M.S.L.
FIG.6 SHT D-25
167
168
169
170
171
172
173
174
18,000
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
1800
1600
1400
1200
1000
800
600
400
200
Q=17200 @ELEV 175.9
PMF DISCHARGE C.F.S.
Inflow Time Hydrograph for Harris Pond
outflow time
HARRIS POND DAM PMF ROUTING THRU RESERVOIR AND OVER DIKE
Surcharge Storage Elev.
Discharge Elev.
SURCHARGE STORAGE A.F.
0
2
4
6
8
10
12
14
TIME HOURS



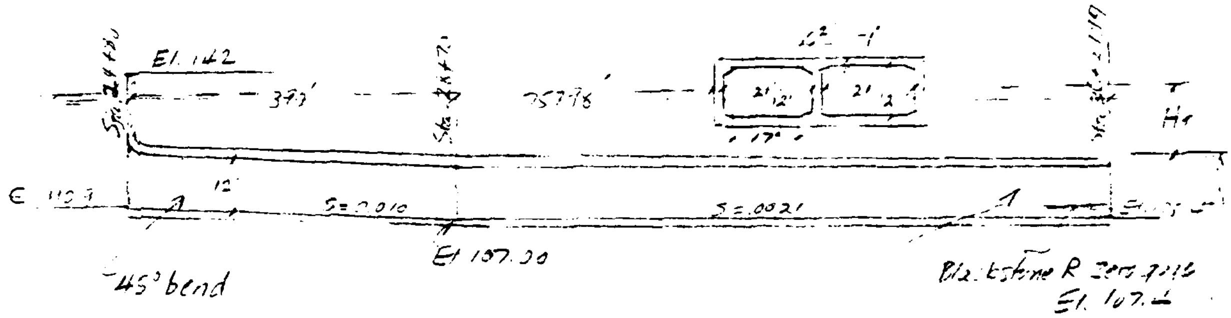
0.5 PMF DISCHARGE CFS
8000
7000
6000
5000
4000
3000
2000
1000
0
0
4
6
8
10
12
14
TIME HOURS
Inflow into Harris Pond
Inflow into Harris Pond
Inflow into Forge
Outflow from Hiawatha
Inflow into Muddy Brook
Inflow into Mill Reservoir


HARRIS POND DAM
0.5 PMF INFLOW HYDROGRAPH
Inflow between Hopedale & North Pond
Outflow from North Pond
Outflow from Hopedale
servoir
Outflow from North Pond
Inflow into unnamed
14
16
18
20
22
24
26
HOURS

BY: [illegible] DATE: 1-29-78 **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. D-27 OF

CHKD. BY: DATE: INSPECTION OF DAMS - CONN & R.I. PROJECT:

SUBJECT: HARRIS POND DAM - MILL RIVER CONDUIT [illegible] HARRIS POND



-45° Bend.

Blackstone R Stage

For free flow in downstream leg of conduit

$S = 0.0021$ $S^{1/2} = .0458$ $n = 0.014$

d	[illegible]	Σ Area	W.P	r	$r^{2/3}$	v	Q	Σ both barrels
10	[illegible]	17	21.07	[illegible]	[illegible]	4.47	85	170
5	94	130	29.07	3.50	2.30	11.10	1152	2305
[illegible]	[illegible]	208.5	37.07	5.27	3.03	14.79	3.064	6128
[illegible]	21	229.0	41.07	5.57	3.17	15.20	3480	6960
12	19	248.0	45.42	5.39	3.18	14.97	3712	7424 Max free flow capacity
Full		248.0	62.92	3.94	2.49			

Pressure conduit flow Entrance loss 0.1 hv 45° Bend loss 0.05 hv

$L = 1148$ $n = 0.014$ $h_f = SL = L\left(\frac{vn}{1.486\,r^{2/3}}\right)^2$ $H_T = 1.2h_v + SL$

Total Q	Q per barrel	A	v	hv	1.2hv	$r^{2/3}$	S	SL	H
8500	4250	248	17.14	4.56	5.5	2.49	.00042	4.8	10.3
10000	5000	248	20.16	6.31	7.6	2.49	.0058	6.7	14.3
12000	6000	248	24.2	9.1	10.9	2.49	.0084	9.6	20.5
13000	6500	248	26.21	10.67	12.8	2.49	.0098	11.3	24.1
11000	5500	248	22.18	7.64	9.2	2.49	.0070	8.1	17.3

Blackstone River Stage - Discharge below Mill River Culvert. NED [illegible] Sheet 102

Zero gage El. 107.4

Q in Blackstone	Stage	Elev.
8500	10.0	117.4
10000	11.0	118.4
12000	12.3	119.7
15000	14.3	121.7
18000	16.0	123.4
21000	17.6	125.0
24000	18.9	126.3

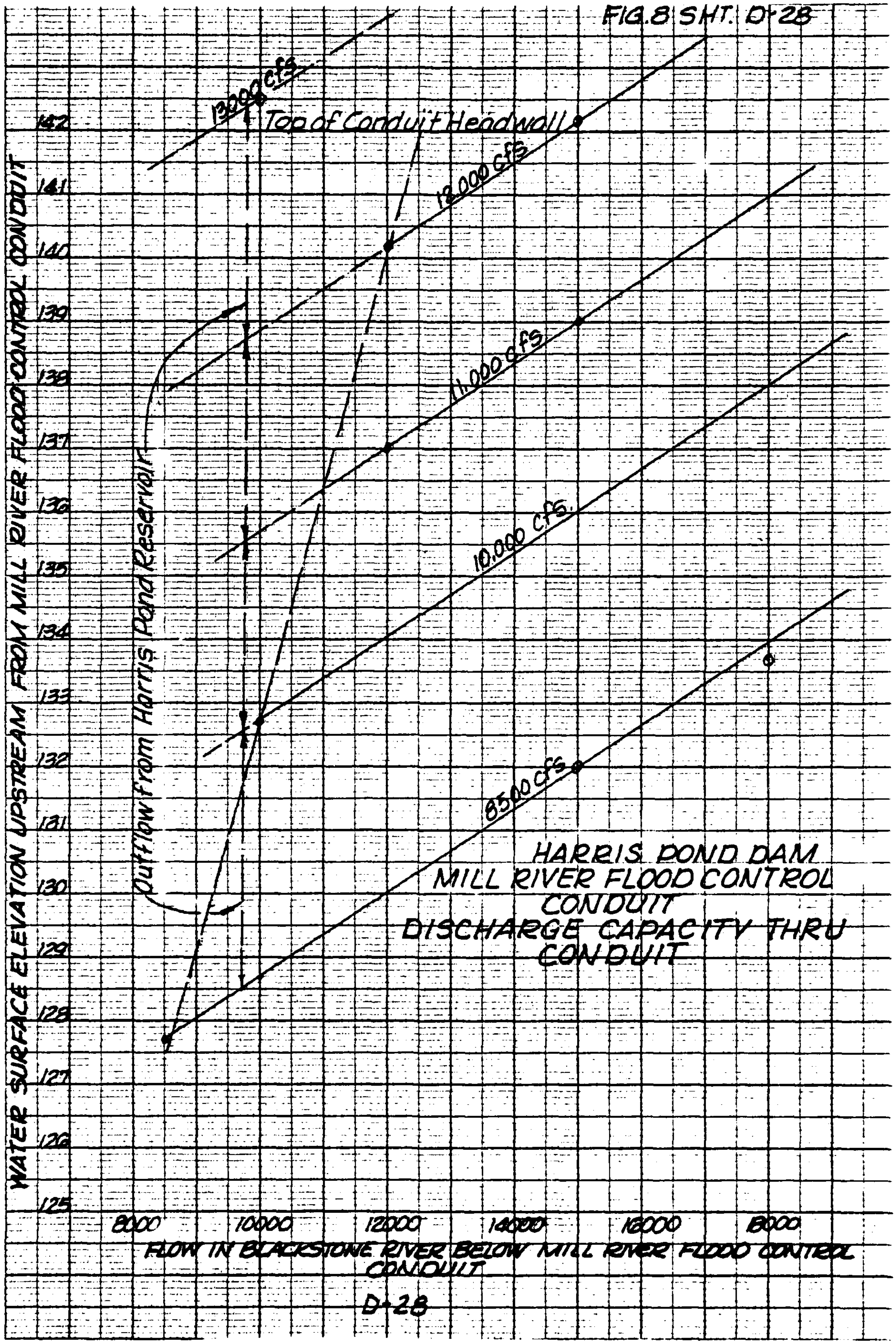


FIG.8 SHT. D-28
13000 cfs
Top of Conduit Headwall
12,000 cfs
11,000 cfs
10,000 cfs.
8500 cfs
Outflow from Harris Pond Reservoir
HARRIS POND DAM
MILL RIVER FLOOD CONTROL
CONDUIT
DISCHARGE CAPACITY THRU
CONDUIT
WATER SURFACE ELEVATION UPSTREAM FROM MILL RIVER FLOOD CONTROL CONDUIT
142
141
140
139
138
137
136
135
134
133
132
131
130
129
128
127
126
125
8000
10000
12000
14000
16000
18000
FLOW IN BLACKSTONE RIVER BELOW MILL RIVER FLOOD CONTROL CONDUIT

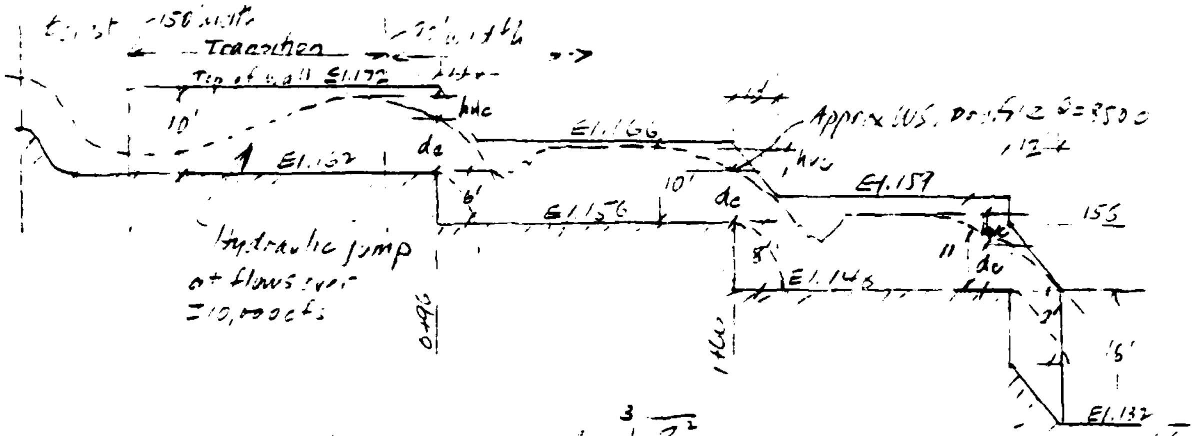
BY ______ DATE ______

LOUIS BERGER & ASSOCIATES INC.

SHEET NO. D-29 OF ______

CHKD. BY ______ DATE ______ INSPECTION OF DAMS - CONN. & R.I. PROJECT ______

SUBJECT HARRIS POND DAM SPILLWAY - HYDRAULICS



For 90' width - critical flow $d_c = \sqrt[3]{\frac{q^2}{g}}$

Q	q (90' width) cfs	d_c	h_{vc}	$d_c + h_{vc}$	Remarks
8500	94.4	6.50	3.25	9.75	Water surface below wall - not considering wave action or air entrainment.
9000	100.0	6.75	3.38	10.13	Overtops walls 0.13' (Not including air & waves)
10,000	111.1	7.25	3.63	10.88	" " 0.88 "
11,000	122.2	7.72	3.86	11.58	" " 1.58 "
12,000	133.3	8.19	4.10	12.29	" " 2.29 "
14,400	160.0	9.24	4.62	13.86	" " 3.86 "
17,200	191.1	10.40	5.20	15.60	" " 5.60 "

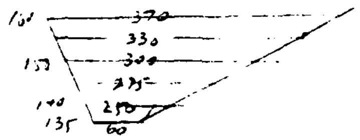
BY JJT DATE 2-13-79 **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. D-30 OF

CHKD. BY ____ DATE ____ INSPECTION OF DAMS CONN. & R.I. PROJECT ____

SUBJECT HARRIS POND DAM - RIVER CHANNEL BELOW DAM - HYDRAULICS

WATERWAY BELOW PRIVILEDGE BRIDGE -
BRIDGE AND ABUTMENT FILL WASHED AWAY

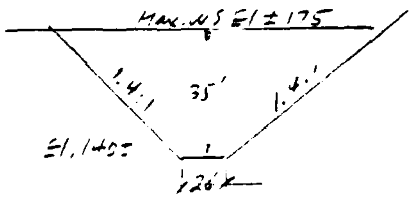
SECTION 500' Downstream from bridge. (No backwater influence from Flood control conduit downstr.)



Critical flow $\frac{A^3}{T} = \frac{Q^2}{g}$

W.S. Elev.	ΔA	ΣA	T	Q_c	V_c	h_{vc}	Backwater El. up st.
140	775	775	250	7743	9.99	1.55	141.5
145	1312	2087	275	32624	15.63	3.79	148.8
150	1438	3525	300	68565	19.45	5.87	155.9

BREACH FAILURE OF DAM.

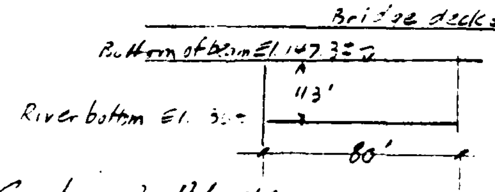


Q thru 20' W $= 1.68 \times 20 \times 35^{3/2} = 6957$ cfs

Q thru ∇ = 17045

24,000 cfs

PRIVILEGE BRIDGE - FLOW THRU WATERWAY

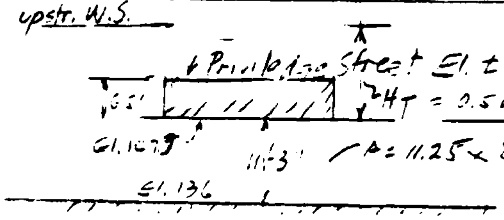


$v_c = \sqrt{g\,d_c}$

Critical Flows.

d_c-ft	A_c	v_c	Q_c	$d_c + h_v$	El. of Backwater upstr. from bridge
5	400	12.69	5075	7.5	143.5
7	560	15.01	8406	10.5	146.5
9	720	17.02	12254	13.5	149.5
11	880	18.82	16562	16.5	152.5

CONDUIT FLOWS UNDER BRIDGE (No overtopping flows)



$H_T = 0.5 h_v + h_v = 1.5 h_v$

$A = 11.25 \times 80 = 900$ sq. ft.

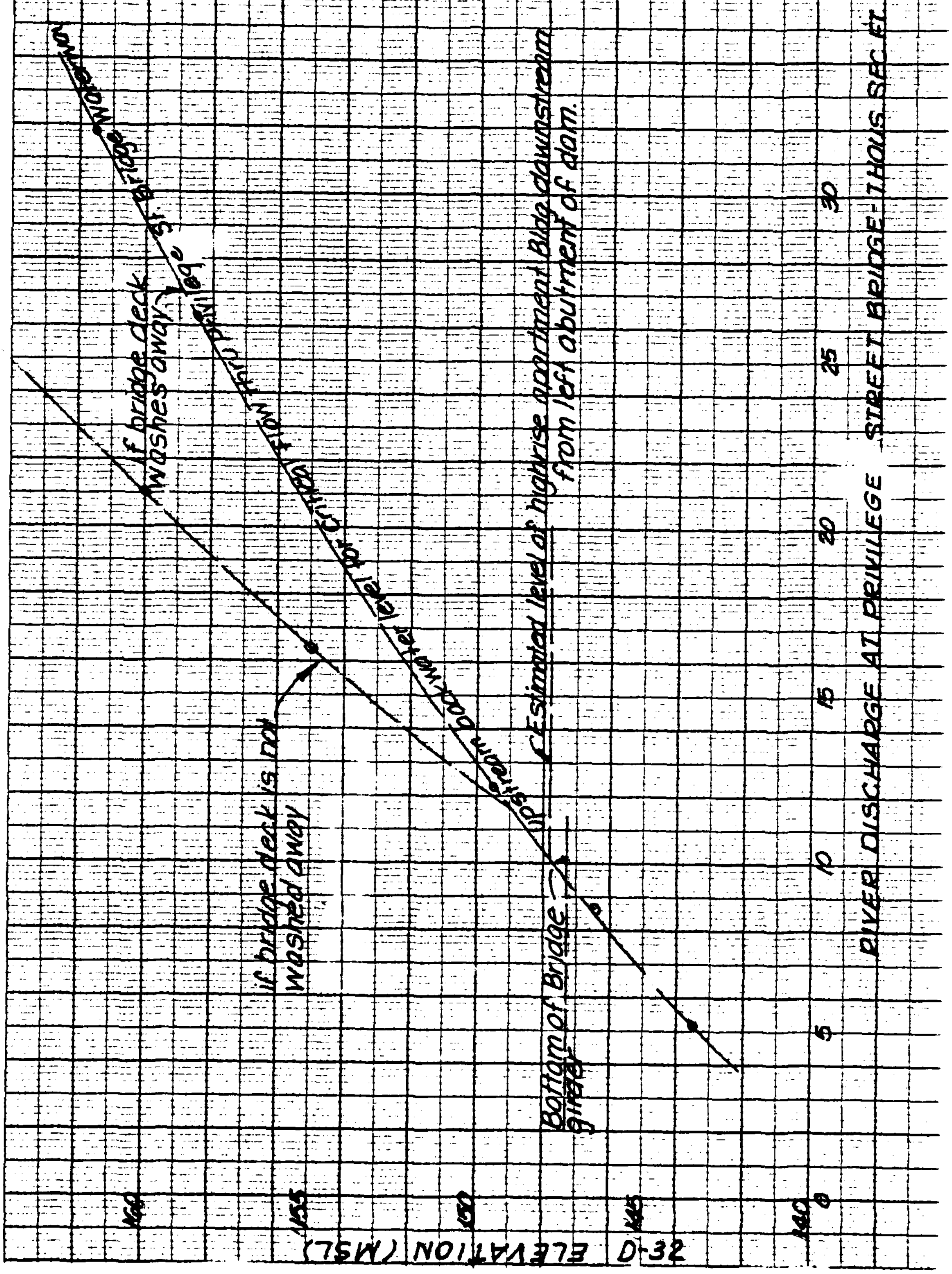
upstr. WSE	H_T	h_v	v	Q
155	7.7	5.13	18.2	16384
160	12.7	8.47	23.36	21020
165	17.7	11.80	27.57	24800
170	22.7	15.13	31.22	28[illegible]

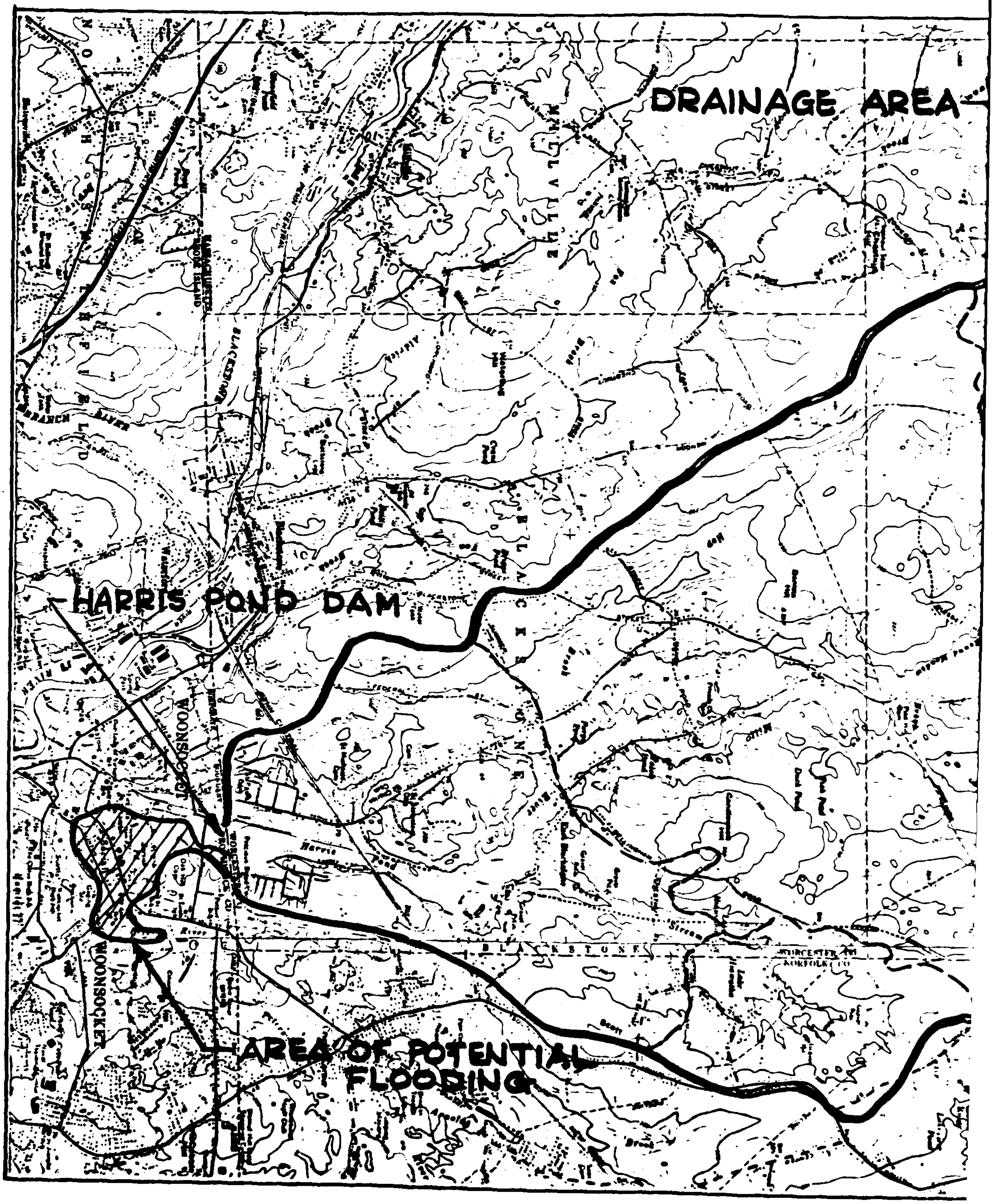
CRITICAL FLOWS THRU 80' WATERWAY IF BRIDGE DECK WASHES AWAY

d_c	A_c	v_c	Q_c	$d_c + h_v$	Elev. of backwater upstr. from bridge
13	1040	20.46	21278	19.5	155.5
15	1200	21.98	26,376	22.5	158.5
17	1360	23.40	31824	25.5	161.5
19	1520	24.73	37590	28.5	164.5
21	1680	26.00	43056	31.5	167.5

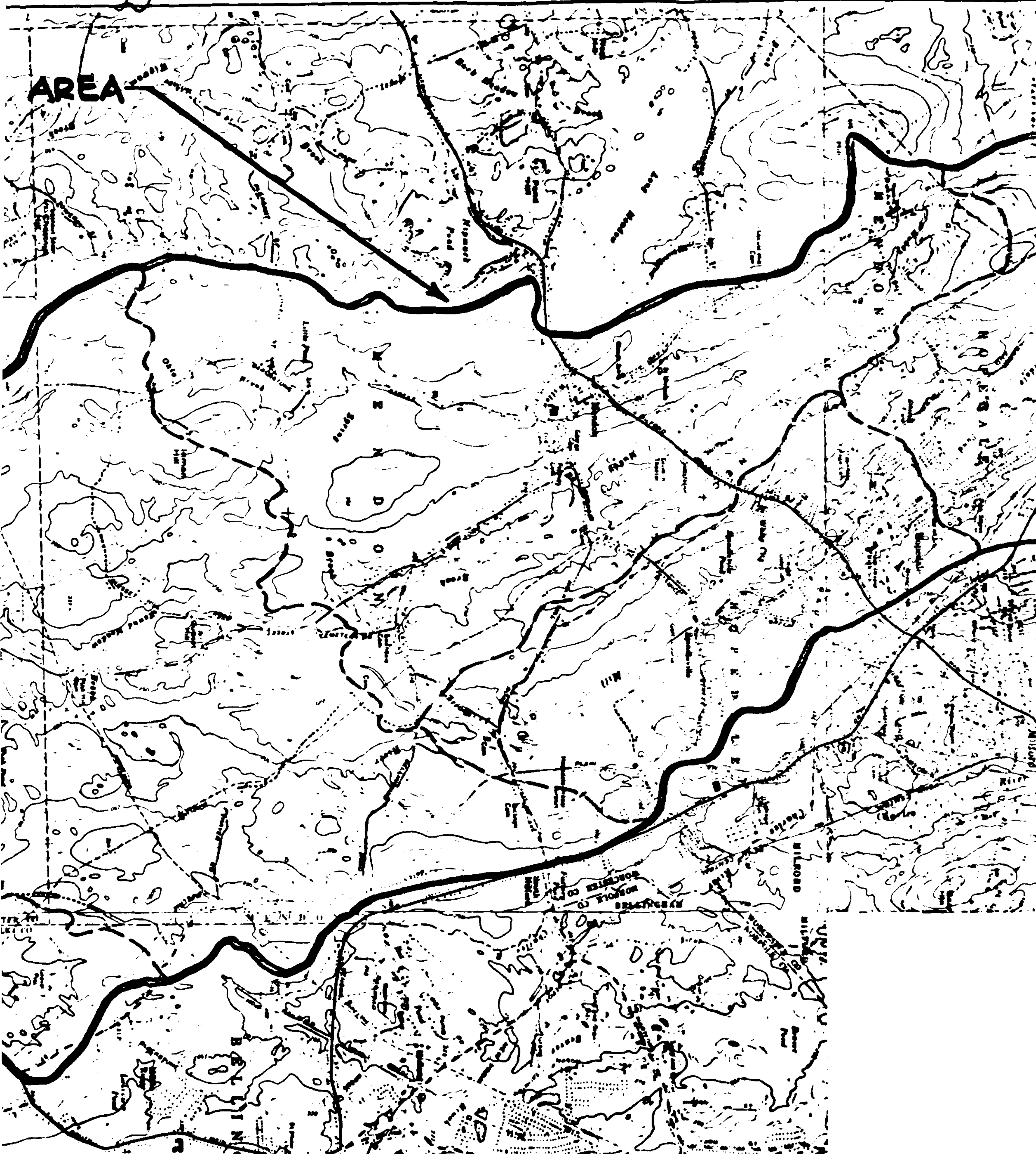
FIG 9 SHT. D-32

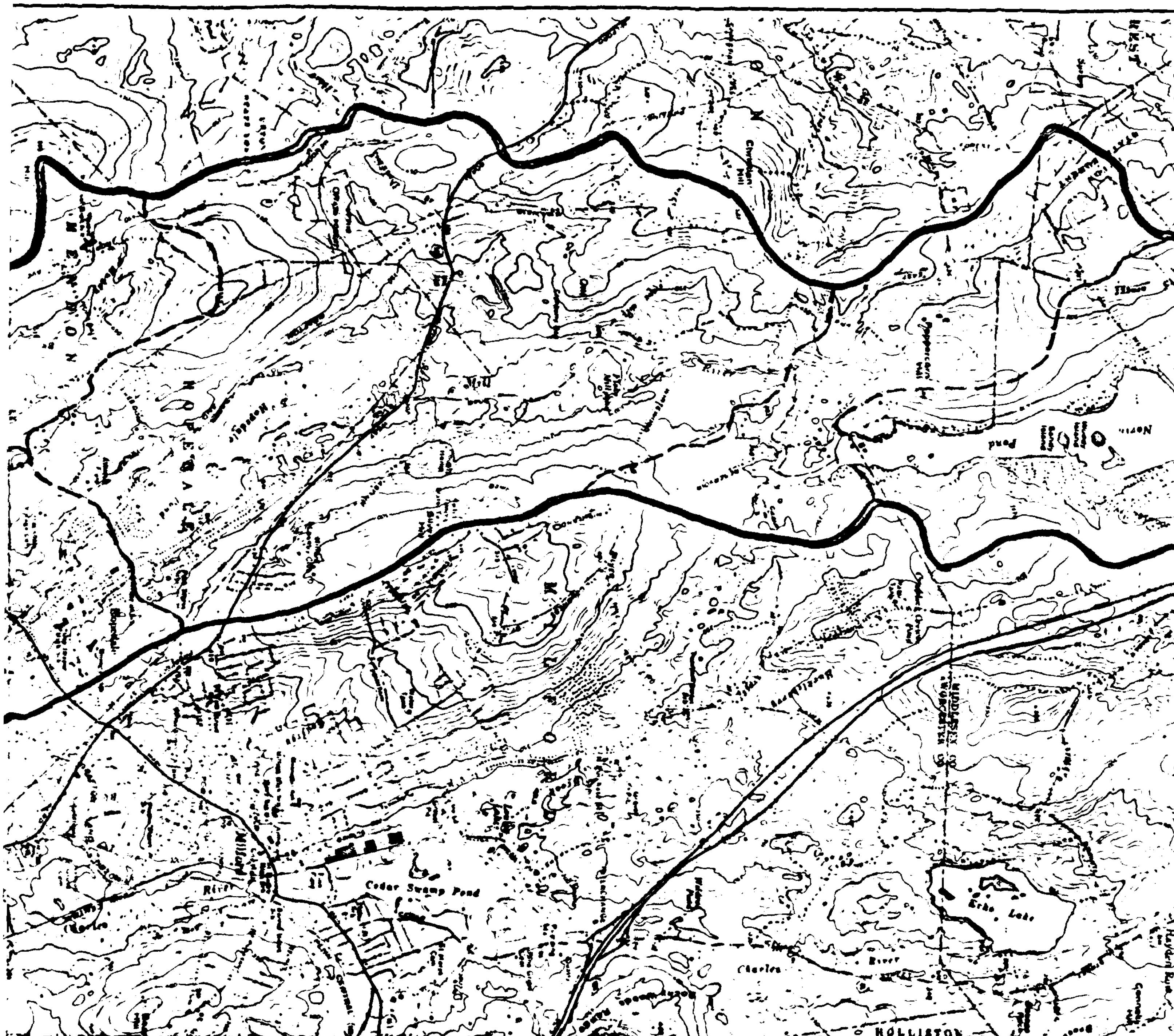
HARRIS POND DAM
ESTIMATED STAGE DISCHARGES ABOVE PRIVILEGE BRIDGE
700 FT. DOWNSTREAM FROM DAM




DRAINAGE AREA
HARRIS POND DAM
AREA OF POTENTIAL FLOODING
WOONSOCKET
BLACKSTONE

2

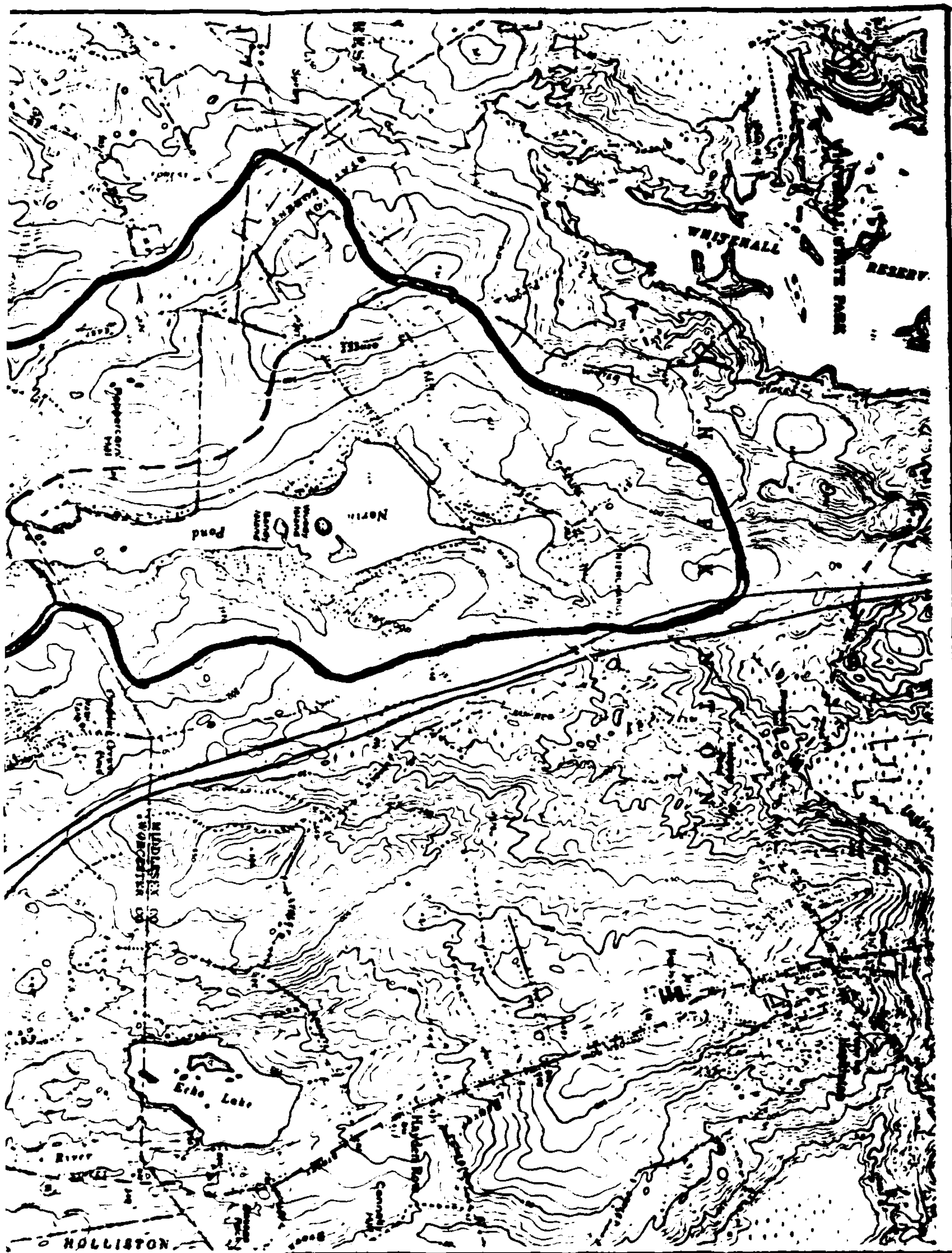




3

LOUIS BER
WELLE
ARCHITEC

NATIONAL



LOUIS BERGER & ASSOC., INC WELLESLEY, MASS. ARCHITECT ENGINEER	US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

HARRIS POND DAM

DRAINAGE AREA AND AREA OF POTENTIAL FLOODING

4

STATE - RI

SCALE

DATE

FIG. 10. - SHEET D-33

[redacted]

```
AREA 1
JAN 79

                         JOB SPECIFICATION
  NQ    NHR   NMIN   IDAY   IHR   IMIN  METRC   IPLT   IPRT  NSTAN
 150      0     30      0     0      0      0    200      0      0
                      JOPER    NWT
                          5      0
```

FULL PMF

```
**********         **********         **********         **********         **********

                         SUB-AREA RUNOFF COMPUTATION

INFLOW
           ISTAQ    ICOMP    IECON    ITAPE     JPLT     JPRT    INAME
               1        0        0        0        0        0        1

                                HYDROGRAPH DATA
  IHYDG    IUHG    TAREA     SNAP    TRSDA    TRSPC    RATIO    ISNOW    ISAME    LOCAL
      0      -1     2.70      0.0     2.70      0.0      0.0        0        0        0

                                PRECIP DATA
                         SPFE      PMS       R6       R12      R24      R48      R72      R96
                          NP    STORM      DAJ      DAK
                          40      0.0      0.0      0.0
                                PRECIP PATTERN
 0.40     0.40 0.89     1.07     1.16     1.34     1.70     4.94     1.16     1.16
 0.98     0.80

                                LOSS DATA
  STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
    0.0     0.0    1.00     0.0     0.0    1.00     0.0     0.0     0.0     0.0

                        GIVEN UNIT GRAPH, NUHGQ=  48
    0.       0.       0.       0.       0.       0.       0.       0.       0.       0.
    0.       0.       0.       0.       0.       0.       0.       0.       0.       0.
    0.       0.       0.       0.       0.       0.       0.       0.      75.     281.
  549.     653.     575.     431.     294.     209.     147.     100.      68.      49.
   34.      23.      16.      12.       8.       6.       3.       1.
         UNIT GRAPH TOTALS   3534. CFS OR 1.01 INCHES OVER THE AREA
```

D-33A

H.P. 1.

TIME	RAIN	EXCS.	COMP Q.
1	0.40	0.40	0.
2	0.40	0.40	0.
3	0.89	0.89	0.
4	1.07	1.07	0.
5	1.16	1.16	0.
6	1.34	1.34	0.
7	1.70	1.70	0.
8	4.94	4.94	0.
9	1.16	1.16	0.
10	1.16	1.16	0.
11	0.98	0.98	0.
12	0.80	0.80	0.
13	0.0	0.0	0.
14	0.0	0.0	0.
15	0.0	0.0	0.
16	0.0	0.0	0.
17	0.0	0.0	0.
18	0.0	0.0	0.
19	0.0	0.0	0.
20	0.0	0.0	0.
21	0.0	0.0	0.
22	0.0	0.0	0.
23	0.0	0.0	0.
24	0.0	0.0	0.
25	0.0	0.0	0.
26	0.0	0.0	0.
27	0.0	0.0	0.
28	0.0	0.0	0.
29	0.0	0.0	30.
30	0.0	0.0	142.
31	0.0	0.0	399.
32	0.0	0.0	811.
33	0.0	0.0	1367.
34	0.0	0.0	1997.
35	0.0	0.0	2641.
36	0.0	0.0	3541.
37	0.0	0.0	4816.
38	0.0	0.0	6105.

TIME	RAIN	EXCS	COMP Q
39	0.0	0.0	6580.
40	0.0	0.0	6232.
41	0.0	0.0	5468.
42	0.0	0.0	4505.
43	0.0	0.0	3520.
44	0.0	0.0	2578.
45	0.0	0.0	1807.
46	0.0	0.0	1251.
47	0.0	0.0	879.
48	0.0	0.0	609.
49	0.0	0.0	419.
50	0.0	0.0	290.
51	0.0	0.0	203.
52	0.0	0.0	137.
53	0.0	0.0	93.
54	0.0	0.0	57.
55	0.0	0.0	33.
56	0.0	0.0	17.
57	0.0	0.0	9.
58	0.0	0.0	3.
59	0.0	0.0	1.
SUM	16.00	16.00	56541.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	6580.	4149.	1178.	393.	56544.
INCHES		14.30	16.23	16.23	16.23
AC-FT		2059.	2338.	2338.	2338.

HP. 2.

13	0.0	0.0	[illegible]		48	0.0	0.0	609.
14	0.0	0.0	0.		49	0.0	0.0	19.
15	0.0	0.0	0.		50	0.0	0.0	296.
16	0.0	0.0	0.		51	0.0	0.0	203.
17	0.0	0.0	0.		52	0.0	0.0	137.
18	0.0	0.0	0.		53	0.0	0.0	93.
19	0.0	0.0	0.		54	0.0	0.0	57.
20	0.0	0.0	0.		55	0.0	0.0	33.
21	0.0	0.0	0.		56	0.0	0.0	17.
22	0.0	0.0	0.		57	0.0	0.0	9.
23	0.0	0.0	0.		58	0.0	0.0	3.
24	0.0	0.0	0.		59	0.0	0.0	1.
25	0.0	0.0	0.					
26	0.0	0.0	0.					
27	0.0	0.0	0.					
28	0.0	0.0	0.		SUM	16.00	16.00	56541.
29	0.0	0.0	30.					
30	0.0	0.0	142.					
31	0.0	0.0	399.					
32	0.0	0.0	811.					
33	0.0	0.0	1367.					
34	0.0	0.0	1597.					
35	0.0	0.0	2141.					
36	0.0	0.0	3541.					
37	0.0	0.0	4816.					
38	0.0	0.0	6105.					

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	6580.	4149.	1178.	393.	56544.
INCHES		14.30	16.23	16.23	16.23
AC-FT		2059.	2338.	2338.	2338.

HP. 2.

HP3.

HYDROGRAPH ROUTING

ROUTING

ISTAG	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
11	1	0	0	0	0	1

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME
0.0	0.0	0.0	1	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA
1	0	0	0.0	0.0	0.0	0.

STORAGE	0.	120.	248.	380.	530.	680.	846.	1010.	1198.	1588.
OUTFLOW	33.	101.	186.	287.	896.	1977.	3329.	4913.	8669.	

D-35

HP3.

HP4

TIME	EOP STOR	AVG IN	EOP OUT
	0.	0.	0.
1	1.	15.	0.
2	4.	86.	1.
3	15.	271.	4.
4	40.	605.	11.
5	84.	1089.	23.
6	152.	1682.	50.
7	245.	2319.	99.
8	367.	3091.	178.
9	530.	4179.	287.
10	725.	5460.	1190.
11	911.	6342.	2511.
12	1048.	6406.	3651.
13	1126.	5850.	4303.
14	1150.	4987.	4506.
15	1132.	4013.	4360.
16	1086.	3049.	3971.
17	1024.	2193.	3444.
18	956.	1529.	2884.
19	892.	1065.	2355.
20	835.	744.	1903.
21	784.	514.	1573.
22	740.	354.	1284.
23	702.	246.	1038.
24	670.	170.	854.
25	642.	115.	740.
26	616.	75.	637.
27	594.	45.	545.
28	574.	25.	465.
29	557.	13.	395.
30	542.	6.	335.
31	529.	2.	286.
32	517.	0.	278.
33	506.	0.	271.
34	495.	0.	263.
35	484.	0.	256.
36	474.	0.	249.
37	464.	0.	242.
38	454.	0.	236.
39	444.	0.	229.
40	435.	0.	223.
41	426.	0.	217.
42	417.	0.	211.
43	408.	0.	205.
44	400.	0.	199.

TIME	EOP STOR	AVG IN	EOP OUT
45	392.	0.	194.
46	384.	0.	189.
47	376.	0.	184.
48	369.	0.	179.
49	361.	0.	174.
50	354.	0.	169.
51	347.	0.	165.
52	341.	0.	161.
53	334.	0.	156.
54	328.	0.	152.
55	322.	0.	148.
56	316.	0.	144.
57	310.	0.	141.
58	304.	0.	137.
59	298.	0.	133.
60	293.	0.	130.
61	288.	0.	126.
62	282.	0.	123.
63	277.	0.	120.
64	272.	0.	117.
65	268.	0.	114.
66	263.	0.	111.
67	259.	0.	108.
68	254.	0.	105.
69	250.	0.	102.
70	246.	0.	100.
71	242.	0.	98.
72	238.	0.	96.
73	234.	0.	93.
74	230.	0.	91.
75	226.	0.	89.
76	223.	0.	87.
77	219.	0.	86.
78	215.	0.	84.
79	212.	0.	82.
80	209.	0.	80.
81	205.	0.	78.
82	202.	0.	77.
83	199.	0.	75.
84	196.	0.	73.
85	193.	0.	72.
86	190.	0.	70.
87	187.	0.	69.
88	184.	0.	67.
89	182.	0.	66.
90	179.	0.	64.
91	176.	0.	63.
92	174.	0.	62.
93	171.	0.	60.
94	169.	0.	59.

SUM 52488.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	4506.	3062.	1013.	365.	52488.
INCHES		10.55	13.97	15.07	15.07
AC-FT		1519.	2011.	2170.	2170.

*********** *********** *********** *********** ***********

SUB-AREA RUNOFF COMPUTATION

HYDROGRAPH FOR UNNAMED BROOK

ISTAG	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
2	0	0	0	0	0	1

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
6	-1	1.70	0.0	1.70	0.0	0.0	0	0	0

PRECIP DATA

NP	STORM	DAJ	DAK
38	0.0	0.0	0.0

PRECIP PATTERN

0.40	0.40	0.89	1.07				
1.16	1.34	1.70	4.94	1.16	1.16	0.98	0.80

LOSS DATA

STRKR	DLTKR	RTIOL	ERAIN	STRKS	RTIOK	STRTL	CNSTL	ALSMX	RTIMP
0.0	0.0	1.00	0.0	0.0	1.00	0.0	0.0	0.0	0.0

GIVEN UNIT GRAPH, NUHGQ= 54

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	6.	24.	51.	86.
132.	170.	195.	206.	199.	181.	159.	136.	112.	94.
79.	66.	56.	46.	38.	32.	26.	22.	19.	15.
13.	11.	9.	7.						

UNIT GRAPH TOTALS 2192. CFS OR 1.00 INCHES OVER THE AREA

D-37

HP5

HP 6

RECESSION DATA

STRTQ= 0.0 QRCSN= 0.0 RTIOR= 1.00

TIME	RAIN	EXCS	COMP Q
1	0.0	0.0	0.
2	0.0	0.0	2.
3	0.0	0.0	12.
4	0.0	0.0	35.
5	0.0	0.0	83.
6	0.0	0.0	166.
7	0.0	0.0	290.
8	0.0	0.0	459.
9	0.0	0.0	694.
10	0.0	0.0	1001.
11	0.0	0.0	1354.
12	0.0	0.0	1749.
13	0.0	0.0	2123.
14	0.0	0.0	2437.
15	0.0	0.0	2641.
16	0.0	0.0	2720.
17	0.0	0.0	2660.
18	0.0	0.0	2493.
	0.0	0.0	2259.
19	0.0	0.0	1988.
20	0.0	0.0	1703.
21	0.0	0.0	1447.
22	0.0	0.0	1213.
23	0.0	0.0	1020.
24	0.0	0.0	854.
25	0.0	0.0	711.
26	0.0	0.0	592.
27	0.0	0.0	495.
28	0.0	0.0	410.
29	0.0	0.0	340.
30	0.0	0.0	283.
31	0.0	0.0	228.
32	0.0	0.0	185.
33	0.0	0.0	148.
34	0.0	0.0	114.
35	0.0	0.0	83.
36	0.0	0.0	40.
37	0.0	0.0	26.
38	0.0	0.0	14.
39	0.0	0.0	6.
40	0.0	0.0	0.
41	0.0	0.0	0.
SUM	16.00	16.00	35075.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	2720.	2130.	731.	244.	35072.
INCHES		11.66	15.99	15.99	15.99
AC-FT		1057.	1450.	1450.	1450.

*********** *********** *********** *********** ***********

COMBINE HYDROGRAPHS

COMBINED WITH NORTH POND

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
0	2	0	0	0	0	1

SUM OF 2 HYDROGRAPHS AT 0

						0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	2.	12.	35.	83.	166.	291.	463.	705.
1024.	1405.	1839.	2301.	2724.	3831.	5232.	6311.	6796.	6764.
6348.	5674.	4890.	4103.	3375.	2757.	2264.	1676.	1533.	1264.
1080.	920.	773.	650.	543.	449.	369.	318.	297.	277.
262.	249.	242.	236.	229.	223.	217.	211.	205.	199.
194.	189.	184.	179.	174.	169.	165.	161.	156.	152.
148.	144.	141.	137.	133.	130.	126.	123.	120.	117.
114.	111.	108.	105.	102.	100.	98.	96.	93.	91.
89.	87.	86.	84.	82.	80.	78.	77.	75.	73.
72.	70.	69.	67.	66.	64.	63.	62.	60.	59.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	6796.	4900.	1722.	606.	87558.
INCHES		10.36	14.56	15.43	15.43
AC-FT		2431.	3417.	3620.	3620.

HP 7.

SUB-AREA RUNOFF COMPUTATION

INFLOW TO HOPEDALE POND

ISTAG	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
3	0	0	0	0	0	1

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
0	-1	5.60	0.0	5.60	0.0	0.0	0	0	0

PRECIP DATA

NP	STORM	DAJ	DAK
32	0.0	0.0	0.0

PRECIP PATTERN

0.40	0.40	0.89	1.07	1.16	1.34	1.70	4.94	1.16	1.16
0.98	0.80								

LOSS DATA

STRKR	DLTKR	RTIOL	ERAIN	STRKS	RTIOK	STRTL	CNSTL	ALSMX	RTIMP
0.0	0.0	1.00	0.0	0.0	1.00	0.0	0.0	0.0	0.0

GIVEN UNIT GRAPH, NUHGQ= 48

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
20.	80.	169.	291.	435.	562.	644.	678.	654.	596.
524.	447.	368.	308.	259.	217.	183.	152.	127.	105.
85.	72.	61.	51.	44.	38.	31.	24.		

UNIT GRAPH TOTALS 7225. CFS OR 1.00 INCHES OVER THE AREA

D-40

HP8

RECESSION DATA

STRTQ= 0.0 QRCSN= 0.0 RTIOR= 1.00

HP9

END-OF-PERIOD FLOW

TIME	RAIN	EXCS	COMP Q
1	0.40	0.40	0.
2	0.40	0.40	0.
3	0.89	0.89	0.
4	1.07	1.07	0.
5	1.16	1.16	0.
6	1.34	1.34	0.
7	1.70	1.70	0.
8	4.94	4.94	0.
9	1.16	1.16	0.
10	1.16	1.16	0.
11	0.98	0.98	0.
12	0.80	0.80	0.
13	0.0	0.0	0.
14	0.0	0.0	0.
15	0.0	0.0	0.
16	0.0	0.0	0.
17	0.0	0.0	0.
18	0.0	0.0	0.
19	0.0	0.0	0.
20	0.0	0.0	0.
21	0.0	0.0	8.
22	0.0	0.0	40.
23	0.0	0.0	117.
24	0.0	0.0	277.
25	0.0	0.0	550.
26	0.0	0.0	958.
27	0.0	0.0	1518.
28	0.0	0.0	2293.
29	0.0	0.0	3308.
30	0.0	0.0	4473.
31	0.0	0.0	5741.
32	0.0	0.0	7300.
33	0.0	0.0	8040.
34	0.0	0.0	8707.
35	0.0	0.0	8958.
36	0.0	0.0	8755.
37	0.0	0.0	8207.
38	0.0	0.0	7433.
39	0.0	0.0	6536.
40	0.0	0.0	5598.
41	0.0	0.0	4752.
42	0.0	0.0	4003.
43	0.0	0.0	3351.
44	0.0	0.0	2803.
45	0.0	0.0	2340.
46	0.0	0.0	1955.
47	0.0	0.0	1628.
48	0.0	0.0	1351.
49	0.0	0.0	1123.
50	0.0	0.0	932.
51	0.0	0.0	761.
52	0.0	0.0	620.
53	0.0	0.0	500.
54	0.0	0.0	388.
55	0.0	0.0	283.
56	0.0	0.0	136.
57	0.0	0.0	89.
58	0.0	0.0	48.

SUM 16.00 16.00 115599.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	8958.	7017.	2408.	835.	115599.
INCHES		11.66	16.00	16.00	16.00
AC-FT		5481.	4779.	4779.	4779.

********** ********** ********** ********** **********

COMBINE HYDROGRAPHS

COMBINE HOPEDALE WITH SUM OF ABOVE

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
0	2	0	0	0	0	1

SUM OF 2 HYDROGRAPHS AT 0

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
2.	40.	117.	277.	550.	958.	1518.	2295.	3308.	4473.
5741.	7000.	8042.	8719.	8994.	8839.	8373.	7723.	6999.	6303.
5776.	5408.	5190.	5103.	5064.	5786.	6860.	7662.	7919.	7696.
7109.	6294.	5390.	4491.	3657.	2893.	2373.	1925.	1552.	1264.
1080.	920.	773.	650.	543.	449.	369.	318.	297.	277.
262.	249.	242.	236.	229.	223.	217.	211.	205.	199.
194.	189.	184.	179.	174.	169.	165.	161.	156.	152.
148.	144.	141.	137.	133.	130.	126.	123.	120.	117.
114.	111.	108.	105.	102.	100.	98.	96.	93.	91.
89.	87.	86.	84.	82.	80.	78.	77.	75.	73.
72.	70.	69.	67.	66.	64.	63.	62.	60.	59.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	8594.	7326.	4072.	1411.	203157.
INCHES		6.82	15.15	15.75	15.75
AC-FT		3635.	8081.	8399.	8399.

HP 10

********** ********** ********** ********** **********

SUB-AREA RUNOFF COMPUTATION

INFLOW HYDROGRAPH

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
4	0	0	0	0	0	1

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
0	-1	2.80	0.0	2.80	0.0	0.0	0	0	0

PRECIP DATA

NP	STORM	DAJ	DAK
22	0.0	0.0	0.0

PRECIP PATTERN

0.40	0.40	0.89	1.07	1.16	1.34	1.70	4.94	1.16	1.16
0.98	0.80								

LOSS DATA

STRKR	DLTKR	RTIOL	ERAIN	STRKS	RTIOK	STRTL	CNSTL	ALSMX	RTIMP
0.0	0.0	1.00	0.0	0.0	1.00	0.0	0.0	0.0	0.0

GIVEN UNIT GRAPH, NUHGQ= 44

4.	20.	43.	76.	117.	163.	209.	241.	263.	271.
266.	249.	228.	203.	179.	152.	133.	114.	100.	87.
76.	65.	57.	49.	42.	35.	31.	27.	23.	20.
18.	16.	14.	12.						

UNIT GRAPH TOTALS 3603. CFS OR 1.00 INCHES OVER THE AREA

HP 11

D-43

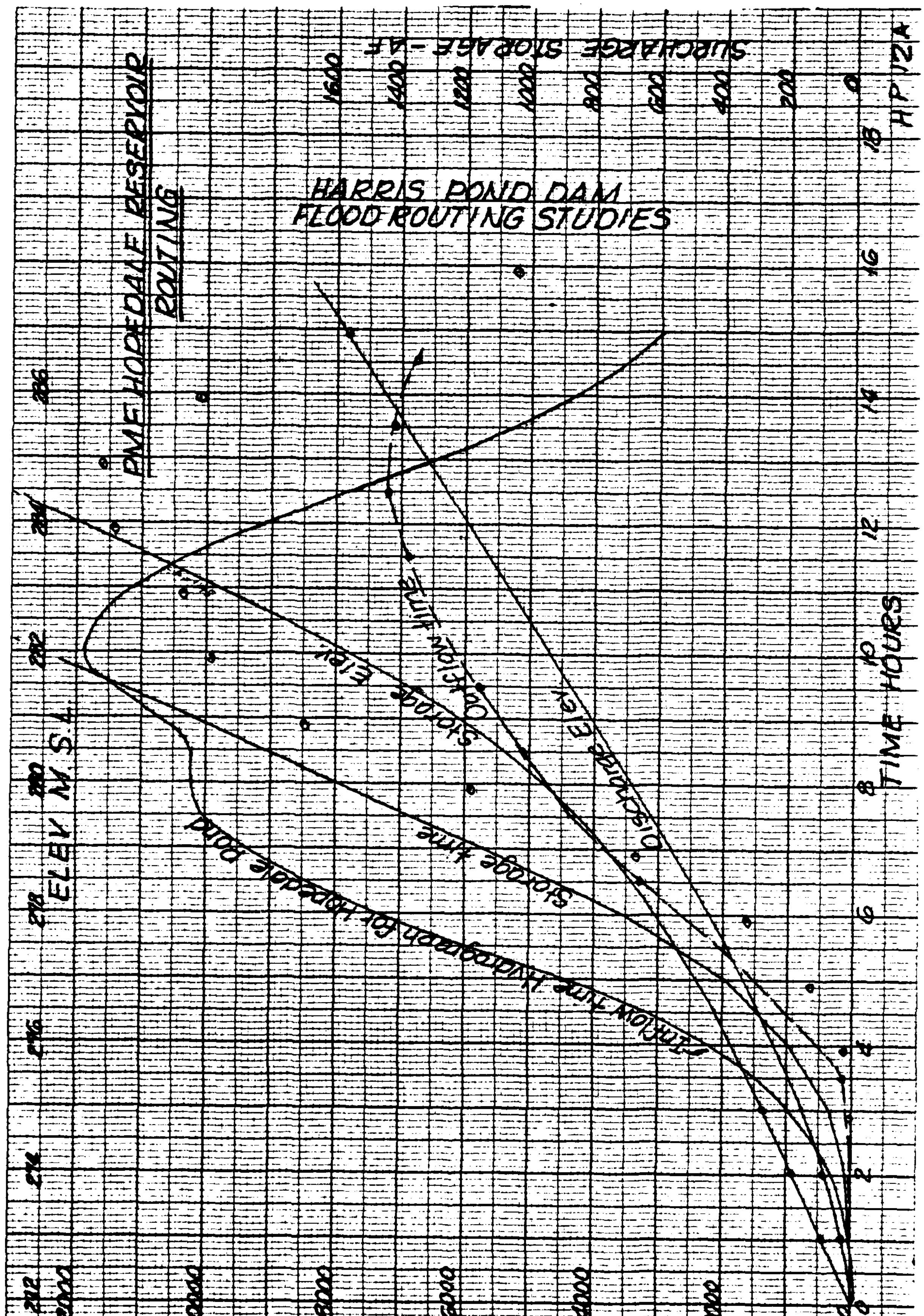
HP 12

END-OF-PERIOD FLOW

TIME	RAIN	EXCS	COMP Q
1	0.40	0.40	0.
2	0.40	0.40	0.
3	0.89	0.89	0.
4	1.07	1.07	0.
5	1.16	1.16	0.
6	1.34	1.34	0.
7	1.70	1.70	0.
8	4.94	4.94	0.
9	1.16	1.16	0.
10	1.16	1.16	0.
11	0.98	0.98	2.
12	0.80	0.80	10.
13	0.0	0.0	29.
14	0.0	0.0	70.
15	0.0	0.0	142.
16	0.0	0.0	254.
17	0.0	0.0	418.
18	0.0	0.0	650.
19	0.0	0.0	976.
20	0.0	0.0	1367.
21	0.0	0.0	1819.
22	0.0	0.0	2304.
23	0.0	0.0	2788.
24	0.0	0.0	3221.
25	0.0	0.0	3539.
26	0.0	0.0	3738.
27	0.0	0.0	3793.
28	0.0	0.0	3727.
29	0.0	0.0	3537.
30	0.0	0.0	3280.
31	0.0	0.0	2972.
32	0.0	0.0	2651.
33	0.0	0.0	2321.
34	0.0	0.0	2033.
35	0.0	0.0	1763.
36	0.0	0.0	1533.
37	0.0	0.0	1329.
38	0.0	0.0	1153.
39	0.0	0.0	996.
40	0.0	0.0	865.
41	0.0	0.0	746.
42	0.0	0.0	643.
43	0.0	0.0	551.
44	0.0	0.0	480.
45	0.0	0.0	411.
46	0.0	0.0	351.
47	0.0	0.0	296.
48	0.0	0.0	250.
49	0.0	0.0	207.
50	0.0	0.0	167.
51	0.0	0.0	128.
52	0.0	0.0	60.
53	0.0	0.0	40.
54	0.0	0.0	23.
55	0.0	0.0	10.

SUM 16.00 16.00 57648.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	3799.	3156.	1201.	400.	57648.
INCHES		10.49	15.96	15.96	15.96
AC-FT		1566.	2383.	2383.	2383.



KEUFFEL & ESSER CO
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K+E STANDARD CROSS SECTION
10 X 10 TO THE HALF INCH

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***********         ***********         ***********         ***********         ***********

                         SUB-AREA RUNOFF COMPUTATION

   HYDROGRAPH FOR MUDDY RIVER
                  ISTAQ    ICOMP    IECON    ITAPE    JPLT    JPRT    INAME
                      5        0        0        0       0       0        1

                                 HYDROGRAPH DATA
   IHYDG    IUHG    TAREA    SNAP    TRSDA    TRSPC    RATIO    ISNOW    ISAME    LOCAL
       0      -1     6.10     0.0     6.10      0.0      0.0        0        0        0

                                   PRECIP DATA
                           NP    STORM      DAJ      DAK
                           22      0.0      0.0      0.0
                                  PRECIP PATTERN

    0.40      0.40      0.89      1.07      1.16      1.34      1.70      4.94      1.16      1.16
    0.98      0.80

                                    LOSS DATA
   STRKR    DLTKR    RTIOL    ERAIN    STRKS    RTIOK    STRTL    CNSTL    ALSMX    RTIMP
     0.0      0.0     1.00      0.0      0.0     1.00      0.0      0.0      0.0      0.0

                          GIVEN UNIT GRAPH, NUHGQ=   44
      0.        0.        0.        0.        0.        0.        0.        0.        0.        0.
      9.       44.       94.      165.      254.      354.      455.      526.      573.      590.
    579.      543.      496.      443.      390.      331.      289.      248.      218.      189.
    165.      142.      124.      106.       92.       77.       67.       58.       51.       44.
     40.       35.       30.       26.
            UNIT GRAPH TOTALS    7847. CFS OR 1.00 INCHES OVER THE AREA
```

D-46

HP 13.

RECESSION DATA

STRTQ= 0.0 QRCSN= 0.0 RTIOR= 1.00

END-OF-PERIOD FLOW

TIME	RAIN	EXCS	COMP Q
1			
2	0.98	0.98	4.
3	0.80	0.80	21.
4	0.0	0.0	63.
5	0.0	0.0	152.
6	0.0	0.0	309.
7	0.0	0.0	554.
8	0.0	0.0	91[illegible].
9	0.0	0.0	141[illegible].
10	0.0	0.0	2127.
11	0.0	0.0	2978.
12	0.0	0.0	3958.
13	0.0	0.0	5014.
14	0.0	0.0	6067.
15	0.0	0.0	7015.
16	0.0	0.0	7710.
17	0.0	0.0	8141.
18	0.0	0.0	8275.
19	0.0	0.0	8117.
20	0.0	0.0	7708.
21	0.0	0.0	7141.
22	0.0	0.0	6476.
23	0.0	0.0	5773.
24	0.0	0.0	5054.
25	0.0	0.0	4423.
26	0.0	0.0	3838.
27	0.0	0.0	3338.
28	0.0	0.0	2890.
29	0.0	0.0	2509.
30	0.0	0.0	2169.
31	0.0	0.0	1882.
32	0.0	0.0	1623.
33	0.0	0.0	1404.
34	0.0	0.0	1205.
35	0.0	0.0	1044.
36	0.0	0.0	895.
37	0.0	0.0	770.
38	0.0	0.0	649.
39	0.0	0.0	548.
40	0.0	0.0	453.
41	0.0	0.0	363.
42	0.0	0.0	278.
43	0.0	0.0	131.
44	0.0	0.0	88.
45	0.0	0.0	49.
46	0.0	0.0	21.

SUM 16.00 16.00 125555.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	8275.	6874.	2616.	872.	125551.
INCHES		10.48	15.96	15.96	15.96
AC-FT		3410.	5191.	5191.	5191.

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**********          **********          **********          **********          **********

                         SUB-AREA RUNOFF COMPUTATION

          INFLOW TO FORGE POND
                       ISTAQ    ICOMP    IECON    ITAPE     JPLT     JPRT    INAME
                         6        0        0        0         0        0        1

                                        HYDROGRAPH DATA
         IHYDG     IUHG    TAREA     SNAP     TRSDA    TRSPC    RATIO    ISNOW    ISAME    LOCAL
            0        -1     9.40      0.0      9.40      0.0      0.0        0        0        0

                                          PRECIP DATA
                                    NP    STORM      DAJ      DAK
                                    12     0.0       0.0      0.0
                                          PRECIP PATTERN
  0.40      0.40      0.69      1.07      1.16      1.34      1.70      4.94      1.16      1.16
  0.98      0.80

                                           LOSS DATA
        STRKR    DLTKR    RTIOL    ERAIN    STRKS    RTIOK    STRTL    CNSTL    ALSMX    RTIMP
          0.0      0.0     1.00      0.0      0.0     1.00      0.0      0.0      0.0      0.0

                                  GIVEN UNIT GRAPH, NUHGQ=   34
   14.       68.      146.      255.      391.      546.      701.      810.      883.      910.
  892.      837.      764.      682.      601.      510.      446.      382.      337.      291.
  255.      218.      191.      164.      141.      118.      104.       89.       79.       68.
   61.       54.       47.       40.
              UNIT GRAPH TOTALS   12095. CFS OR 1.00 INCHES OVER THE AREA
```

D-48

HP.15

RECESSION DATA

STRTQ= 0.0 QRCSN= 0.0 RTIOR= 1.00

END-OF-PERIOD FLOW

TIME	RAIN	EXCS	COMP Q				
1	0.40	0.40	6.	23	0.0	0.0	7789.
2	0.40	0.40	33.	24	0.0	0.0	6819.
3	0.89	0.89	98.	25	0.0	0.0	5915.
4	1.07	1.07	236.	26	0.0	0.0	5149.
5	1.16	1.16	477.	27	0.0	0.0	4454.
6	1.34	1.34	856.	28	0.0	0.0	3869.
7	1.70	1.70	1404.	29	0.0	0.0	3340.
8	4.94	4.94	2185.	30	0.0	0.0	2901.
9	1.16	1.16	3281.	31	0.0	0.0	2503.
10	1.16	1.16	4595.	32	0.0	0.0	2160.
11	0.98	0.98	6105.	33	0.0	0.0	1853.
12	0.80	0.80	7728.	34	0.0	0.0	1612.
13	0.0	0.0	9353.	35	0.0	0.0	1378.
14	0.0	0.0	10610.	36	0.0	0.0	1188.
15	0.0	0.0	11879.	37	0.0	0.0	1000.
16	0.0	0.0	12545.	38	0.0	0.0	843.
17	0.0	0.0	12754.	39	0.0	0.0	699.
18	0.0	0.0	12537.	40	0.0	0.0	563.
19	0.0	0.0	11379.	41	0.0	0.0	429.
20	0.0	0.0	11003.	42	0.0	0.0	203.
21	0.0	0.0	9978.	43	0.0	0.0	136.
22	0.0	0.0	8897.	44	0.0	0.0	77.
				45	0.0	0.0	32.
				46	[illegible]	[illegible]	[illegible]

SUM 16.00 16.00 193521.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	12754.	10593.	4032.	1344.	193519.
INCHES		10.48	15.96	15.96	15.96
AC-FT		5256.	8001.	8001.	8001.

HP16

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**********          **********          **********          **********          **********

                            SUB-AREA RUNOFF COMPUTATION

          INFLOW TO HARRIS POND
                       ISTAO   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME
                           6       0       0       0       0       0       1

                                       HYDROGRAPH DATA
          IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
              0      -1    3.10     0.0    3.10     0.0     0.0       0       0       0

                                         PRECIP DATA
                                   NP   STORM      DAJ      DAK
                                   12     0.0      0.0      0.0
                                       PRECIP PATTERN
   0.40     0.40     0.89     1.07     1.16     1.34     1.70     4.94     1.16     1.16
   0.98     0.80

                                          LOSS DATA
         STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
           0.0     0.0    1.00     0.0     0.0    1.00     0.0     0.0     0.0     0.0

                              GIVEN UNIT GRAPH, NUHGQ=   15
   190.     720.    1000.     810.     510.     320.     200.     115.      75.      45.
    30.      18.      11.       5.       2.
               UNIT GRAPH TOTALS    4051. CFS OR 1.01 INCHES OVER THE AREA
```

D-50

HP 17.

HP-18

RECESSION DATA

STRTQ= 0.0 QRCSN= 0.0 RTIOR= 1.00

END-OF-PERIOD FLOW

TIME	RAIN	EXCS	COMP Q
1	0.40	0.40	76.
2	0.40	0.40	364.
3	0.89	0.89	657.
4	1.07	1.07	1568.
5	1.16	1.16	2409.
6	1.34	1.34	3213.
7	1.70	1.70	3976.
8	4.94	4.94	5399.
9	1.16	1.16	7751.
10	1.16	1.16	8792.
11	0.98	0.98	7930.
12	0.80	0.80	6561.
13	0.0	0.0	5336.
14	0.0	0.0	3947.
15	0.0	0.0	2574.
16	0.0	0.0	1612.
17	0.0	0.0	990.
18	0.0	0.0	613.
19	0.0	0.0	369.
20	0.0	0.0	225.
21	0.0	0.0	127.
22	0.0	0.0	70.
23	0.0	0.0	33.
24	0.0	0.0	16.
SUM	16.00	16.00	64816.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	8792.	4958.	1350.	450.	64816.
INCHES		14.88	16.21	16.21	16.21
AC-FT		2460.	2680.	2680.	2680.

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**********          **********          **********          **********          **********

                         SUB-AREA RUNOFF COMPUTATION

          INFLOW FROM HIAWATHA
                    ISTAQ    ICOMP    IECON    ITAPE    JPLT    JPRT    INAME
                        7        0        0        0       0       0        1

                                   HYDROGRAPH DATA
          IHYDG    IUHG    TAREA    SNAP    TRSDA    TRSPC    RATIO    ISNOW    ISAME    LOCAL
              0      -1     1.10     0.0     1.10      0.0      0.0        0        0        0

                                   PRECIP DATA
                              NP    STORM      DAJ      DAK
                              15      0.0      0.0      0.0
                                   PRECIP PATTERN
   0.40      0.40      0.89      1.07      1.16      1.34      1.70      4.94      1.16      1.16
   0.98      0.80

                                   LOSS DATA
   STRKR    DLTKR    RTIOL    ERAIN    STRKS    RTIOK    STRTL    CNSTL    ALSMX    RTIMP
     0.0      0.0     1.00      0.0      0.0     1.00      0.0      0.0      0.0      0.0

                              GIVEN UNIT GRAPH, NUHGQ=  22
     0.        0.        0.       31.      114.      224.      266.      234.      176.      120.
    85.       60.       41.       28.       20.       14.        9.        7.        5.        3.
     2.        1.
          UNIT GRAPH TOTALS   1440. CFS OR 1.01 INCHES OVER THE AREA
```

D-52

HP 19

HP 20

RECESSION DATA

STRTQ= 0.0 QRCSN= 0.0 RTIOR= 1.00

END-OF-PERIOD FLOW

TIME	RAIN	EXCS	COMP Q
1	0.40	0.40	0.
2	0.40	0.40	0.
3	0.89	0.89	0.
4	1.07	1.07	12.
5	1.16	1.16	58.
6	1.34	1.34	163.
7	1.70	1.70	331.
8	4.94	4.94	557.
9	1.16	1.16	814.
10	1.16	1.16	1077.
11	0.98	0.98	1445.
12	0.80	0.80	1961.
13	0.0	0.0	2489.
14	0.0	0.0	2681.
15	0.0	0.0	2339.
16	0.0	0.0	2230.
17	0.0	0.0	1838.
18	0.0	0.0	1435.
19	0.0	0.0	1052.
20	0.0	0.0	739.
21	0.0	0.0	512.
22	0.0	0.0	359.
23	0.0	0.0	249.
24	0.0	0.0	169.
25	0.0	0.0	119.
26	0.0	0.0	82.
27	0.0	0.0	53.
28	0.0	0.0	35.
29	0.0	0.0	21.
30	0.0	0.0	10.
31	0.0	0.0	6.
32	0.0	0.0	3.
33	0.0	0.0	1.
SUM	16.00	16.00	23040.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	2681.	1692.	480.	160.	23040.
INCHES		14.31	16.24	16.24	16.24
AC-FT		839.	953.	953.	953.

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HYDROGRAPH ROUTING

HIAWATHA

ROUTING THROUGH RESERVOIR

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
77	1	0	0	0	0	1

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME
0.0	0.0	0.0	1	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA
1	0	0	0.0	0.0	0.0	0.

STORAGE 0.	62.	128.	198.	273.	353.	437.	0.	0.	0.
OUTFLOW 0.	574.	2144.	4530.	7587.	11166.	15553.	0.	0.	0.

HP21

HP 22.

TIME	EOP STOR	AVG IN	EOP OUT
1	0.	0.	0.
2	0.	0.	0.
3	0.	0.	0.
4	0.	6.	2.
5	1.	35.	13.
6	5.	110.	44.
7	12.	247.	109.
8	23.	444.	217.
9	40.	686.	367.
10	60.	945.	553.
11	80.	1261.	997.
12	99.	1703.	1462.
13	120.	2225.	1965.
14	136.	2585.	2432.
15	141.	2610.	2579.
16	136.	2385.	2418.
17	127.	2034.	2109.
18	113.	1636.	1798.
19	98.	1244.	1432.
20	83.	895.	1079.
21	71.	625.	780.
22	61.	435.	564.
23	52.	304.	480.
24	42.	209.	393.
25	34.	144.	313.
26	26.	101.	245.
27	20.	68.	188.
28	15.	44.	142.
29	11.	28.	105.
30	8.	16.	77.
31	6.	8.	55.
32	4.	4.	38.
33	3.	2.	27.
34	2.	0.	18.
35	1.	0.	12.
36	1.	0.	8.

SUM 23040.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	2579.	1635.	480.	160.	23040.
INCHES		13.62	16.24	16.24	16.24
AC-FT		811.	953.	953.	953.

0. 01

NORTH POND
AREA 1
JAN 75

JOB SPECIFICATION

NQ	NHR	NMIN	IDAY	IHR	IMIN	METRC	IPLT	IPRT	NSTAN
150	0	30	0	0	0	0	0	0	0

0.5 PMF

JOPER	NWT
3	0

***********   ***********   ***********   ***********   ***********

SUB-AREA RUNOFF COMPUTATION

INFLOW

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
1	0	0	0	0	0	1

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
0	-1	2.70	0.0	2.70	0.0	0.500	0	0	0

PRECIP DATA

NP	STORM	DAJ	DAK
40	0.0	0.0	0.0

PRECIP PATTERN

0.40	0.40	0.89	1.07	1.16	1.34	1.70	4.94	1.16	1.16
0.98	0.80								

LOSS DATA

STRKR	DLTKR	RTIOL	ERAIN	STRKS	RTIOK	STRTL	CNSTL	ALSMX	RTIMP
0.0	0.0	1.00	0.0	0.0	1.03	0.0	0.0	0.0	0.0

GIVEN UNIT GRAPH, NUHGQ= 48

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	75.	281.
549.	653.	575.	431.	294.	209.	147.	100.	68.	49.
34.	23.	16.	12.	8.	6.	3.	1.		

UNIT GRAPH TOTALS 3534. CFS OR 1.01 INCHES OVER THE AREA

D-56

HP23

RUNOFF MULTIPLIED BY 0.50

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	15.	71.
199.	406.	684.	999.	1321.	1771.	2408.	3052.	3290.	3116.
2734.	2253.	1760.	1289.	904.	626.	439.	305.	209.	145.
102.	69.	47.	29.	16.	8.	4.	2.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	3290.	2075.	589.	196.	28272.
INCHES		7.15	8.12	8.12	8.12
AC-FT		1029.	1169.	1169.	1169.

D-57

*********** *********** *********** *********** ***********

HYDROGRAPH ROUTING

ROUTING

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME

ROUTING DATA

GLOSS	CLOSS	AVG	IRES	ISAME
0.0	0.0	0.0	1	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA
1	0	0	0.0	0.0	0.0	0.

F= STORAGE	0.	12[illegible].	248.	380.	530.	680.	846.	1010.	1198.	1588.
W= OUTFLOW	0.	33.	101.	186.	287.	896.	1977.	3329.	4913.	6669.

HP 24

HP25

TIME	EOP STOR	AVG IN	EOP OUT
1	0.	0.	0.
2	0.	7.	0.
3	2.	43.	1.
4	8.	135.	2.
5	20.	302.	5.
6	42.	545.	12.
7	76.	841.	21.
8	123.	1160.	35.
9	185.	1546.	67.
10	267.	2089.	113.
11	374.	2730.	182.
12	496.	3171.	264.
13	610.	3203.	612.
14	697.	2925.	1010.
15	752.	2493.	1362.
16	775.	2007.	1515.
17	775.	1525.	1517.
18	760.	1096.	1417.
19	736.	765.	1262.
20	710.	533.	1089.
21	684.	372.	919.
22	658.	257.	809.
23	634.	177.	711.
24	612.	123.	620.
25	592.	85.	537.
26	573.	55.	463.
27	557.	38.	397.
28	543.	23.	339.
29	530.	12.	289.
30	519.	6.	280.
31	508.	3.	272.
32	497.	1.	265.
33	486.	0.	257.
34	475.	0.	250.
35	465.	0.	243.
36	455.	0.	237.
37	446.	0.	230.
38	436.	0.	224.
39	427.	0.	218.
40	418.	0.	212.
41	410.	0.	206.
42	401.	0.	200.
43	393.	0.	195.
44	385.	0.	189.
45	377.	0.	184.
46	370.	0.	179.
47	363.	0.	175.
48	355.	0.	170.
49	348.	0.	166.

SUM 24377.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1517.	1070.	437.	169.	24377.
INCHES		3.69	6.02	7.00	7.0[illegible]

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***********        ***********        ***********        ***********        ***********

                            SUB-AREA RUNOFF COMPUTATION

      HYDROGRAPH FOR UNNAMED BROOK
                     ISTAQ    ICOMP    IECON    ITAPE    JPLT    JPRT    INAME
                        2        0        0        0        0       0        1

                                     HYDROGRAPH DATA
     IHYDG    IUHG    TAREA    SNAP    TRSDA    TRSPC    RATIO    ISNOW    ISAME    LOCAL
        0      -1      1.70     0.0     1.70     0.0     0.500      0        0        0

                                       PRECIP DATA
                             NP    STORM      DAJ      DAK
                             39     0.0       0.0      0.0
                                     PRECIP PATTERN
                                                           0.40     0.40     0.89     1.07
      1.16     1.34     1.70     4.94     1.16     1.16    0.98     0.80

                                        LOSS DATA
         STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
          0.0     0.0    1.00     0.0     0.0    1.00     0.0     0.0     0.0     0.0

                               GIVEN UNIT GRAPH, NUHGQ=  54
        0.       0.       0.       0.       0.       0.       0.       0.       0.       0.
        0.       0.       0.       0.       0.       0.       0.       0.       0.       0.
        0.       0.       0.       0.       0.       0.       6.      24.      51.      86.
      132.     170.     195.     206.     199.     181.     159.     136.     112.      94.
       79.      66.      56.      46.      38.      32.      26.      22.      19.      15.
       13.      11.       9.       7.
             UNIT GRAPH TOTALS   2192. CFS OR 1.00 INCHES OVER THE AREA
```

D-59

HP 26

COMBINE HYDROGRAPHS

COMBINED WITH NORTH POND

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
0	2	0	0	0	0	1

SUM OF 2 HYDROGRAPHS AT 0

					0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	1.	6.	18.	42.	83.	145.	232.	352.
512.	698.	904.	1129.	1332.	1503.	1624.	1942.	2256.	2491.
2509.	2369.	2141.	1872.	1599.	1346.	1164.	1007.	867.	742.
633.	539.	453.	381.	353.	325.	306.	277.	263.	250.
239.	230.	224.	218.	212.	206.	200.	195.	189.	184.
179.	175.	170.	166.	161.	157.	153.	149.	145.	141.
138.	134.	130.	127.	124.	120.	117.	114.	111.	108.
105.	103.	100.	98.	96.	94.	92.	90.	88.	86.
84.	82.	80.	79.	77.	75.	74.	72.	70.	69.
67.	66.	65.	63.	62.	60.	59.	58.	57.	55.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	2509.	1915.	781.	291.	41913.
INCHES		4.65	6.60	7.38	7.38
AC-FT		950.	1550.	1733.	1733.

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HP27

INCHES		11.66	15.99	15.99		15.99		
AC-FT		1057.	1450.	1450.		1450.		

RUNOFF MULTIPLIED BY 0.50

						0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	1.	6.	18.	42.	83.	145.	230.	347.
500.	677.	870.	1061.	1218.	1321.	1360.	1330.	1247.	1129.
594.	652.	723.	609.	510.	427.	355.	296.	247.	205.
170.	141.	114.	92.	74.	57.	41.	20.	13.	7.
3.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1360.	1065.	365.	122.	17536.
INCHES		5.83	8.00	8.00	8.00
AC-FT		528.	725.	725.	725.

....

D-61

HP28

SUB-AREA RUNOFF COMPUTATION

INFLOW TO HOPEDALE POND

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
3	0	0	0	0	0	1

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
0	-1	5.60	0.0	5.60	0.0	0.500	0	0	0

PRECIP DATA

NP	STORM	DAJ	DAK
32	0.0	0.0	0.0

PRECIP PATTERN

0.40	0.40	0.89	1.07	1.16	1.34	1.70	4.94	1.16	1.16
0.98	0.80								

LOSS DATA

STRKR	DLTKR	RTIOL	ERAIN	STRKS	RTIOK	STRTL	CNSTL	ALSMX	RTIMP
0.0	0.0	1.00	0.0	0.0	1.00	0.0	0.0	0.0	0.0

GIVEN UNIT GRAPH, NUHGQ= 48

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
20.	80.	169.	291.	435.	562.	644.	676.	654.	596.
524.	447.	368.	308.	259.	217.	183.	152.	127.	105.
85.	72.	61.	51.	44.	36.	31.	24.		

UNIT GRAPH TOTALS 7225. CFS OR 1.00 INCHES OVER THE AREA

D-62

HP 29

RUNOFF MULTIPLIED BY 0.50

									0.
4.	20.	59.	138.	275.	479.	759.	1147.	1654.	2236.
2871.	3509.	4020.	4354.	4479.	4378.	4103.	3716.	3268.	2799.
2376.	2002.	1676.	1401.	1170.	978.	814.	675.	562.	466.
380.	310.	259.	194.	141.	68.	44.	24.	10.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	4479.	3506.	1204.	401.	57800.
INCHES		5.63	8.00	8.00	8.00
AC-FT		1741.	2390.	2390.	2390.

D-63

HP30

*********** *********** *********** *********** ***********

COMBINE HYDROGRAPHS

COMBINE HOPEDALE WITH SUM OF ABOVE

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
0	2	0	0	0	0	1

SUM OF 2 HYDROGRAPHS AT 0

									0.
4.	20.	55.	138.	275.	479.	759.	1147.	1654.	2236.
2871.	3500.	4021.	4360.	4497.	4419.	4186.	3862.	3500.	3151.
2888.	2700.	2580.	2530.	2502.	2480.	2438.	2618.	2819.	2957.
2889.	2678.	2391.	2066.	1741.	1414.	1208.	1031.	877.	742.
633.	539.	453.	381.	353.	329.	306.	277.	263.	250.
239.	230.	224.	218.	212.	206.	200.	195.	189.	184.
179.	175.	170.	166.	161.	157.	153.	149.	145.	141.
138.	134.	130.	127.	124.	120.	117.	114.	111.	108.
105.	103.	100.	98.	96.	94.	92.	90.	88.	86.
84.	82.	80.	79.	77.	75.	74.	72.	70.	69.
67.	66.	65.	63.	62.	60.	59.	58.	57.	55.

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HP31

HYDROGRAPH ROUTING

ROUTING THROUGH HOPEDALE

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
43	1	0	0	0	0	1

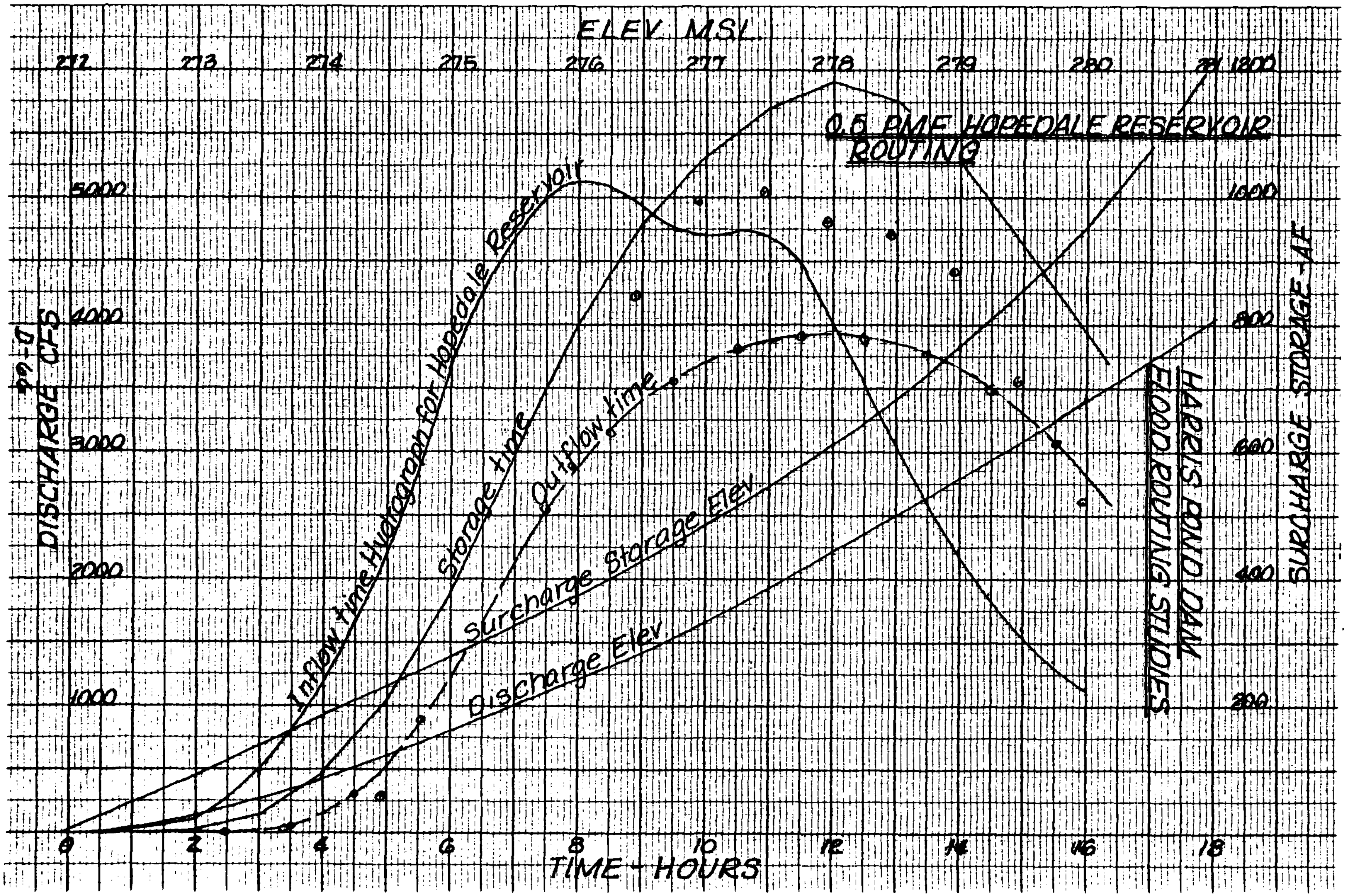
ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME
0.0	0.0	0.0	1	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA
1	0	0	0.0	0.0	0.0	0.

RAGE= STORAGE=0.	89.	180.	274.	374.	481.	612.	762.	957.	1221.
FLOW= OUTFLOW=0.	150.	428.	780.	1210.	1690.	2222.	2800.	3421.	4000.

HP 32


HARRIS POND DAM
FLOOD ROUTING STUDIES
0.5 PMF HOPEDALE RESERVOIR ROUTING
ELEV MSL
272
273
274
275
276
277
278
279
280
281
DISCHARGE CFS
1000
2000
3000
4000
5000
SURCHARGE STORAGE-AF
200
400
600
800
1000
1800
TIME - HOURS
0
2
4
6
8
10
12
14
16
18
Inflow time Hydrograph for Hopedale Reservoir
Storage time
Outflow time
Surcharge Storage Elev
Discharge Elev

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                         SUB-AREA RUNOFF COMPUTATION

        HYDROGRAPH FOR MUDDY RIVER
                     ISTAG   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME
                         5       0       0       0       0       0       1

                                  HYDROGRAPH DATA
        IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
            0      -1    6.10     0.0    6.10     0.0   0.500       0       0       0

                                   PRECIP DATA
                                 NP   STORM     DAJ     DAK
                                 22     0.0     0.0     0.0
                                   PRECIP PATTERN

     0.40    0.40    0.89    1.07    1.16    1.34    1.70    4.94    1.16    1.16
     0.98    0.80

                                   LOSS DATA
        STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
          0.0     0.0    1.00     0.0     0.0    1.00     0.0     0.0     0.0     0.0

                           GIVEN UNIT GRAPH, NUHGQ=  44
       0.      0.      0.      0.      0.      0.      0.      0.      0.      0.
       9.     44.     94.    165.    254.    354.    455.    526.    573.    590.
     579.    543.    496.    443.    390.    331.    289.    248.    218.    189.
     165.    142.    124.    106.     92.     77.     67.     58.     51.     44.
      40.     35.     30.     26.
            UNIT GRAPH TOTALS   7847. CFS OR 1.00 INCHES OVER THE AREA
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HP33

RUNOFF MULTIPLIED BY 0.50

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
2.	11.	32.	76.	154.	277.	455.	708.	1063.	1489.
1979.	2507.	3034.	3507.	3855.	4070.	4137.	4059.	3854.	3570.
3238.	2887.	2527.	2211.	1919.	1669.	1445.	1254.	1085.	941.
812.	702.	602.	522.	448.	385.	324.	274.	226.	182.
139.	66.	44.	25.	10.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	4137.	3437.	1308.	436.	62776.
INCHES		5.24	7.98	7.98	7.98
AC-FT		1705.	2595.	2595.	2595.

HP34.

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**********     **********     **********     **********     **********

                         SUB-AREA RUNOFF COMPUTATION

      INFLOW TO HARRIS POND
                ISTAQ   ICOMP   IECON   ITAPE   JPLT   JPRT   INAME
                    6       0       0       0      0      0       1

                              HYDROGRAPH DATA
     IHYDG   IUHG   TAREA   SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
         0     -1    3.10    0.0    3.10     0.0   0.500       0       0       0

                                PRECIP DATA
                         NP   STORM     DAJ     DAK
                         12     0.0     0.0     0.0
                               PRECIP PATTERN
  0.40    0.40    0.89    1.07    1.16    1.34    1.70    4.94    1.16    1.16
  0.98    0.80

                                 LOSS DATA
    STRKR  DLTKR  RTIOL  ERAIN  STRKS  RTIOK  STRTL  CNSTL  ALSMX  RTIMP
      0.0    0.0   1.00    0.0    0.0   1.00    0.0    0.0    0.0    0.0

                       GIVEN UNIT GRAPH, NUHGG=  15
  190.     720.    1000.    810.     510.     320.     200.     115.     75.     45.
   30.      18.      11.      5.       2.
           UNIT GRAPH TOTALS   4051. CFS OR 1.01 INCHES OVER THE AREA

                               RECESSION DATA
              STRTQ=    0.0     QRCSN=    0.0     RTIOR= 1.00
```

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HP 35

RUNOFF MULTIPLIED BY 0.50

38.	182.	429.	784.	1204.	1606.	1988.	2699.	3875.	4396.
3965.	3281.	2668.	1974.	1287.	806.	495.	307.	185.	113.
64.	35.	17.	8.	3.	1.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	4396.	2479.	675.	225.	32408.
INCHES		7.44	8.10	8.10	8.10
AC-FT		1230.	1340.	1340.	1340.

D-70

HP36

D-71

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***********          ***********          ***********          ***********          ***********

                                SUB-AREA RUNOFF COMPUTATION

         INFLOW HYDROGRAPH
                     ISTAQ    ICOMP    IECON    ITAPE     JPLT     JPRT    INAME
                         4        0        0        0        0        0        1

                                    HYDROGRAPH DATA
         IHYDG     IUHG    TAREA     SNAP    TRSDA    TRSPC    RATIO    ISNOW    ISAME    LOCAL
             0       -1     2.80      0.0     2.80      0.0    0.500        0        0        0

                                      PRECIP DATA
                                 NP    STORM      DAJ      DAK
                                 22      0.0      0.0      0.0
                                    PRECIP PATTERN

    0.40      0.40      0.89      1.07      1.16      1.34      1.70      4.94      1.16      1.16
    0.98      0.80

                                       LOSS DATA
         STRKR    DLTKR    RTIOL    ERAIN    STRKS    RTIOK    STRTL    CNSTL    ALSMX    RTIMP
           0.0      0.0     1.00      0.0      0.0     1.00      0.0      0.0      0.0      0.0

                              GIVEN UNIT GRAPH, NUHGQ=  44
       0.        0.        0.        0.        0.        0.        0.        0.        0.        0.
       4.       20.       43.       76.      117.      163.      209.      241.      263.      271.
     266.      249.      228.      203.      179.      152.      133.      114.      100.       67.
      76.       65.       57.       49.       42.       35.       31.       27.       23.       20.
      18.       16.       14.       12.
            UNIT GRAPH TOTALS   3603. CFS OR 1.00 INCHES OVER THE AREA
```

HP37

RUNOFF MULTIPLIED BY 0.50

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
1.	5.	14.	35.	71.	127.	209.	325.	488.	684.
909.	1152.	1394.	1611.	1769.	1869.	1900.	1863.	1769.	1640.
1486.	1325.	1161.	1016.	881.	767.	664.	577.	498.	432.
373.	322.	276.	240.	206.	176.	146.	125.	104.	83.
64.	30.	20.	11.	5.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1900.	1578.	600.	200.	28824.
INCHES		5.24	7.98	7.98	7.98
AC-FT		783.	1192.	1192.	1192.

D-72

HP 38

SUB-AREA RUNOFF COMPUTATION

INFLOW TO FORGE POND

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
6	0	0	0	0	0	1

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
0	-1	9.40	0.0	9.40	0.0	0.500	0	0	0

PRECIP DATA

NP	STORM	DAJ	DAK
12	0.0	0.0	0.0

PRECIP PATTERN

0.40	0.40	0.89	1.07	1.16	1.54	1.70	4.94	1.16	1.16
0.98	0.80								

LOSS DATA

STRKR	DLTKR	RTIOL	ERAIN	STRKS	RTIOK	STRTL	CNSTL	ALSMX	RTIMP
0.0	0.0	1.00	0.0	0.0	1.00	0.0	0.0	0.0	0.0

GIVEN UNIT GRAPH, NUHGG= 34

14.	68.	144.	255.	391.	546.	701.	810.	883.	910.
892.	837.	764.	682.	601.	510.	446.	382.	337.	291.
255.	218.	191.	164.	141.	118.	104.	89.	79.	68.
61.	54.	47.	40.						

UNIT GRAPH TOTALS 12095. CFS OR 1.00 INCHES OVER THE AREA

D-73

HP39

RUNOFF MULTIPLIED BY 0.50

3.	16.	49.	118.	239.	428.	702.	1092.	1640.	2298.
3052.	3864.	4677.	5405.	5940.	6272.	6377.	6253.	5939.	5501.
4989.	4448.	3894.	3409.	2957.	2575.	2227.	1935.	1670.	1450.
1251.	1080.	927.	806.	689.	594.	500.	422.	349.	282.
214.	101.	68.	38.	16.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	6377.	5297.	2016.	672.	96759.
INCHES		5.24	7.98	7.98	7.98
AC-FT		2628.	4000.	4000.	4000.

HP40

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                         SUB-AREA RUNOFF COMPUTATION

         INFLOW FROM HIAWATHA
                     ISTAQ    ICOMP    IECON    ITAPE    JPLT    JPRT    INAME
                        7        0        0        0        0       0       1

                                   HYDROGRAPH DATA
         IHYDG    IUHG    TAREA    SNAP    TRSDA    TRSPC    RATIO    ISNOW    ISAME    LOCAL
             0      -1     1.10     0.0     1.10      0.0    0.500        0        0        0

                                     PRECIP DATA
                                NP    STORM    DAJ    DAK
                                15     0.0     0.0    0.0
                                    PRECIP PATTERN
    0.0      0.0      0.0      0.40     0.40     0.89     1.07     1.16     1.34     1.70
    4.94     1.16     1.16     0.98     0.80

                                      LOSS DATA
         STRKR    DLTKR    RTIOL    ERAIN    STRKS    RTIOK    STRTL    CNSTL    ALSMX    RTIMP
          0.0      0.0     1.00      0.0      0.0     1.00      0.0      0.0      0.0      0.0

                              GIVEN UNIT GRAPH, NUHGQ=  22
     0.       0.       0.      31.     114.     224.     266.     234.     176.     120.
    85.      60.      41.      28.      20.      14.       9.       7.       5.       3.
     2.       1.
              UNIT GRAPH TOTALS   1440. CFS OR 1.01 INCHES OVER THE AREA

                                    RECESSION DATA
                    STRTQ=     0.0     QRCSN=     0.0     RTIOR= 1.00
```

HP41

RUNOFF MULTIPLIED BY 0.50

0.	0.	0.	0.	0.	0.	0.	25.	81.	165.
279.	407.	538.	722.	980.	1245.	1341.	1270.	1115.	919.
718.	526.	369.	256.	179.	124.	85.	60.	41.	27.
17.	11.	5.	3.	1.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1341.	846.	240.	80.	11520.
INCHES		7.15	8.12	8.12	8.12
AC-FT		420.	476.	476.	476.

D-76

HYDROGRAPH ROUTING

ROUTING THROUGH RESERVOIR

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
77	1	0	0	0	0	1

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME
0.0	0.0	0.0	1	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA
1	0	0	0.0	0.0	0.0	0.

64 = STORAGE	0.	62.	128.	198.	273.	353.	437.	0.	0.	0.
~~244~~ = OUTFLOW	0.	574.	2144.	4530.	7587.	11166.	15553.	0.	0.	0.

HP42

HP43

TIME	EOP STOR	AVG IN	EOP OUT
1	0.	0.	0.
2	0.	3.	1.
3	1.	18.	6.
4	2.	55.	22.
5	6.	123.	55.
6	12.	222.	108.
7	20.	343.	184.
8	30.	473.	276.
9	42.	630.	390.
10	58.	851.	538.
11	75.	1113.	879.
12	86.	1293.	1152.
13	91.	1305.	1253.
14	89.	1192.	1213.
15	83.	1017.	1084.
16	76.	818.	909.
17	68.	622.	720.
18	60.	448.	558.
19	52.	313.	479.
20	43.	218.	395.
21	34.	152.	317.
22	27.	105.	249.
23	21.	72.	192.
24	16.	50.	147.
25	12.	34.	110.
26	9.	22.	82.
27	6.	14.	60.
28	5.	8.	43.
29	3.	4.	31.
30	2.	2.	22.
31	2.	1.	15.
32	1.	0.	10.
33	1.	0.	7.
34	1.	0.	5.
35	0.	0.	3.
36	0.	0.	2.
37	0.	0.	1.

APPENDIX E

INFORMATION AS CONTAINED IN THE

NATIONAL INVENTORY OF DAMS

INVENTORY OF DAMS IN THE UNITED STATES

STATE	IDENTITY NUMBER	DIVISION	STATE	COUNTY	CONGR DIST.	STATE	COUNTY	CONGR DIST.	NAME	LATITUDE (NORTH)	LONGITUDE (WEST)	REPORT DATE DAY MO YR
RI	3001	NED	RI	007	01				HARRIS POND DAM	4200.0	7150.4	01MAR79

POPULAR NAME	NAME OF IMPOUNDMENT
	HARRIS POND

REGION	BASIN	RIVER OR STREAM	NEAREST DOWNSTREAM CITY-TOWN-VILLAGE	DIST FROM DAM (MI.)	POPULATION
01	06	MILL RIVER	WOONSOCKET	0	46820

TYPE OF DAM	YEAR COMPLETED	PURPOSES	STRUCTURAL HEIGHT (FT.)	HYDRAULIC HEIGHT (FT.)	IMPOUNDING CAPACITIES MAXIMUM (ACRE-FT.)	IMPOUNDING CAPACITIES NORMAL (ACRE-FT.)	DIST	OWN	FED R	PRV/FED	SCS A	VER/DATE
RECIPG	1969	S	40	40	2850	1050	NED	N	N	N	N	07MAR79

REMARKS
21 ZONE FILL

D/S HAS	SPILLWAY CREST LENGTH	SPILLWAY TYPE	SPILLWAY WIDTH (FT.)	MAXIMUM DISCHARGE (FT.)	VOLUME OF DAM (CY)	POWER CAPACITY INSTALLED (MW)	POWER CAPACITY PROPOSED (MW)	NAVIGATION LOCKS NO	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)
1	1018	U	150	18500	186296											

OWNER	ENGINEERING BY	CONSTRUCTION BY
CITY OF WOONSOCKET	METCALF + EDDY INC.	CAMPANELLA CORP

REGULATORY AGENCY DESIGN	CONSTRUCTION	OPERATION	MAINTENANCE
NONE	NONE	NONE	NONE

INSPECTION BY	INSPECTION DATE DAY MO YR	AUTHORITY FOR INSPECTION
LOUIS BERGER + ASSOCIATES, INC.	27SEP78	PL92-367

REMARKS

ATE

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-8